SPW

1 SPW

The bulkheads are made up of a relatively thin vertical structure, driven into the ground up to a certain depth underneath the dredge level, so as to obtain a support solid enough to oppose the thrusts of the ground, of water and of possible overloads. This type of supporting structure can be made up of prefabricated and embedded sheet piles, drilled piles, diaphragms in reinforced concrete constructed in site and sometimes also of reinforced concrete panels.

SECTION PILE

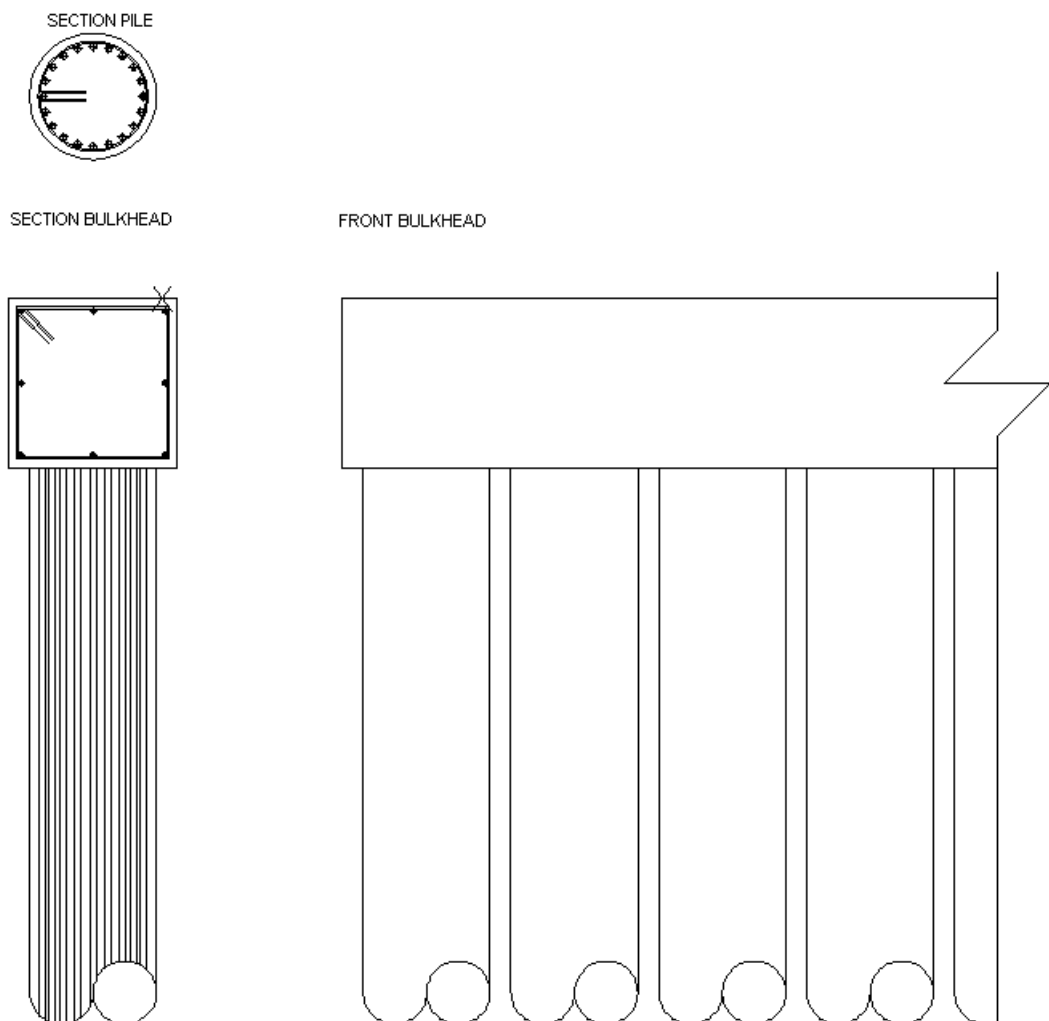SECTION BULKHEAD   FRONT BULKHEAD

Diagram of a bulkhead built by placing reinforced concrete piles close to each other

The most widely used calculation methods are the following:

- *Limit Equilibrium Method* (LEM)
- *Finite-elements Method* (FEM)

While the limit equilibrium method is based on considerations which are only and exclusively of static nature, for the finite-elements method considerations also based on the congruence of the deformations (the FEM method is thus a more rational method). The mentioned methods have a growing complexity both from the numerical point of view and in terms of preliminary operations for the calculation. In fact, while for the LEM method it is necessary to know the classical properties of the soil (angle of internal friction, etc.) for the FEM method it is also necessary to estimate the modulus of subgrade reaction of the ground and characterize its possible non-linear behaviour. The SPW software allows to perform the analysis of cantilever, anchored or braced bulkheads according above mentioned calculation methods.

GENERAL FEATURES (As regards the software input)

SPW software is able to analyze the models that are representative for most of the problems met in current practice. From a general point of view (we will go into details in the following sections), the main features of the input are the following:

- Materials that constitute the bulkhead (Materials archive)
- Sections of the vertical structure (Sections archive)
- Anchorage stringcourses
- Anchors (Anchors archive)
- Management of settings related to the reinforcement of the structure (Reinforcements options)
- Calculation methods (Limit equilibrium, Finite-elements)
- Soil model geometry (treatable both in terms of coordinates of vertices and in terms of angles and distances)
- Soil properties (it is possible to define different soil properties an consider different stratifications)
- It is possible to consider the presence of the groundwater, also for filtration studies and consequently siphoning checks
- Acting loads, treated both by means of concentrated loads and loads divided by stripes, lines or uniform loads
- Analysis phases. It is possible to define different scenarios of cementation of the structure, referring to analysis phases which can change by stratigraphy, materials features, imposed restrictions, loads, etc.

GENERAL FEATURES (As regards the calculation phase)

- Analysis achievable with the limit equilibrium method and the finite-elements method
- Analysis achievable for more loads combinations and for different analysis phases

- Calculation of the pressures due to the presence of the groundwater, external loads, etc.
- Analysis of the stresses on a continuous beam or the head beam or the anchoring beam
- Metrical computation
- Global stability analysis (using the methods: Fellenius, Bishop, Janbu, Bell, Sarma, D.E.M.)
- Bulkhead sections
- Determination of the flow grid and calculation of the filtration flow rate

GENERAL FEATURES (As regards the output phase)
- Display of the horizontal pressures and stresses diagrams
- Results of the bulkhead structural analysis in terms of checks or section design
- Generation of technical reports (including theoretical notes) selective with respect to the subjects to be printed (it is possible to print theoretical notes, input data, calculation output)
- Generation of complete graphic printouts in which are defined the measures, quantities and shaping of the reinforcements resulting from the calculation

1.1   Chiarimenti sulle modalità di calcolo del software SPW

The SPW software allows for analyses using the limit equilibrium method (L.E.M) and the finite element method (F.E.M). The first method can generally be used to perform a simplified calculation, thereby creating a preliminary design of the retaining wall and defining its main characteristics (embedment depth, diameter, etc.). This calculation only considers the static aspect of the problem, neglecting the actual deformability of the soil. Thus, the calculation is independent of the deformation state of the soil-structure complex. In contrast, by using the F.E.M., it is possible to perform both linear and nonlinear analyses, taking into account both the static and kinematic aspects. Therefore, the actual soil-structure interaction is considered, resulting in a more accurate calculation that aligns with the real behavior of the works.

Analysis with the Finite Element Method (F.E.M.)

The finite element method, as highlighted earlier, allows for more realistic analyses by considering the actual interaction between soil and structure. In this approach, the retaining wall is modeled as a set of beams with continuity constraints (beam elements) connected to the soil through

elastic springs, whose stiffness is evaluated based on the elastic properties of the soil (Figure 1).
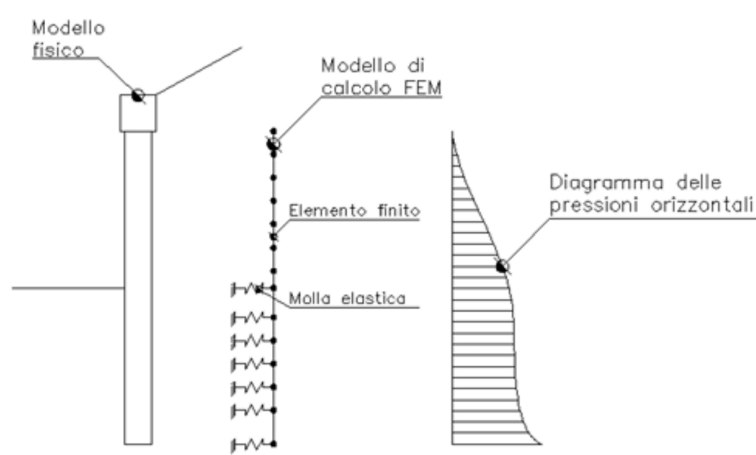
Figure 1: Schematic of a retaining wall using the finite element method

The stiffness calculation of the elastic springs (ks) is defined as the ratio between the ultimate resistance of the soil and the horizontal displacement:

$$k_s = \frac{q_{ult}}{\delta}$$

Such a relationship can be derived from pile load tests, or alternatively, it can be estimated based on formulations used for bearing capacity calculation using the following equation:

$$k_s = A_s + B_s Z^n$$

in where:

$$A_s = C\left(cN_c + 0.5\gamma BN_\gamma\right)$$
$$B_s = C\left(\gamma N_q\right)$$
Z = depth considered

C = 40 obtained from the ratio $\frac{1}{\delta}$ by imposing a displacement δ of 0.0254 m

$$n = p \tan \varphi$$

$$N_q = \exp\left(n\left(\tan^2\left(45° + \frac{\varphi}{2}\right)\right)\right)$$
$$N_c = (Nq - 1)\cot\varphi$$
$$N_\varphi = 1.5(Nq - 1)\tan\varphi$$

Therefore, once the stiffness modulus value of the spring ks, is determined, the software applies a force F at each node of the model calculated using the following relationship:

$$F = k_s \cdot \delta$$

The calculation using this model, as demonstrated by Bowles (1974), can be considered in the calculation of the stresses induced in the region of the retaining wall subjected to passive pressure conditions. The model has been tested on numerous cases reported by Tschebotarioff (1949) and Rowe (1952), yielding satisfactory results.

To summarize, a linear analysis is initially performed where the displacements of the excavation base nodes are checked. If these displacements exceed the specified limit (0.0254) used in the calculation of ks, then a nonlinear analysis is initiated. This approach ensures that the ultimate limit states (ULS) of soil resistance are upheld..

Note (provided by the user on the transition from linear to nonlinear analysis)

1.2   New computation model

SPW offers the possibility to create a new file using a guided procedure. When you click on the command "New" it opens the following window:

Environment for the management of a new model

The start of a new project requires the definition of the following data

Project
Input of a synthetic description of the project: description, location, designer and date. It includes the possibility to print this data in the computation report.

Standards
Select the standards regarding the geotechnical checks (GEO standards) or the structural checks (STR standards). For each category of checks, the following choices are possible:

- GEO standards: Eurocode 7 and Eurocode 8
- STR standards: Eurocode 2

Calculation of pressures

In this group of data are defined the theories which can be used for the calculation of the active thrust coefficient, for the passive thrust coefficient and for the limit state coefficient (active or passive) in dynamic conditions. In particular, the following options are available:

- Active pressures: Coulomb, Muller-Breslau, Caqout-Kerisel, Jaky, Rankine
- Passive pressures: Coulomb, Muller-Breslau, Caqout-Kerisel
- Seismic pressures: Mononobe-Okabe, structures that don't undergo displacements
- Application point of the seismic action to 2/3 H, 1/3 H or 1/2 H.

Computation model

It is possible to select a priori the calculation approach for the determination of the stresses and displacements. In particular, it is possible to choose the LEM method (Limit Equilibrium Method) or the FEM method *(Finite Elements Method)*.

Vertical bearing capacity

Is given the possibility to calculate the vertical bearing capacity (Yes/No) or to indicate the correlation factor of the verified verticals referred to the bearing capacity of the pile.

Longitudinal extension

Indicate the longitudinal extension of the bulkhead.

Geometry

In this group of data are set the data sizes with which to initialize the model. In particular:

- Excavation height: it is expressed in m and represents the part of the bulkhead that remains above ground of after the excavation
- Downstream/Upstream ground inclination: in degrees, they affect the value of active and passive thrust coefficients
- Cross section: choose from among the sections defined in the Sections archive. In this first phase of the project, the chosen section is extended to the whole height of the excavation, but further it is possible to differentiate various sections throughout the height of excavation, assigning different types of section.

1.3   Computation procedure

Once initialized a new model through the "New" command, the user can proceed to assign or change the input data.

As an example you can find bellow a sequence of operations that allow to perform a complete analysis of a bulkhead:

1. Add in the Archive section the geometry of the bulkhead section that you are going to analyze and choose the material
2. Define the reinforcements options
3. Define the structure of the bulkhead which can also be constituted by sections of different type
4. Input ground geometry data
5. Define the soil characteristics and the eventual presence of groundwater
6. Define the characteristics of any struts or anchors
7. Define any loads on the ground and on the structure
8. Set the analysis method (L.E.M. or F.E.M.)
9. In case you have chosen the method F.E.M. define any pressure, reaction modulus and boundary conditions
10. Define the computation combinations for the construction phases that you want to analyze, assign the seismic parameters for seismic combinations, start the analysis of the bulkhead
11. Once the analysis is performed you can view all the results and reinforcements designed (structural analysis results, diagrams, sections carpentry, etc. ..)
12. Export the computation report using the command "Create report" in the Output menu
13. Carry out the analysis of global stability and export the results.

Further will be described the windows that allow the user to perform the analysis.

1.4 Archives

1.4.1 Material archive

To this unique materials archive refer all the sections of the structural elements planned in the programme. Each of the data appearing in the tables by default can be changed (also for the considerations developed afterwards) and do not constitute any vehicle for the designer as the only person responsible for the values assumed.

NB: *In order to delete any kind of conglomerate amongst the ones expounded in the table, you only have to delete all the characters present in the box of Concrete Class column.*

The environment through which it is possible to mange the materials archive is the following:

CONGLOMERATES

N°	Concrete class	fck,cubes [MPa]	Ec [MPa]	fck [MPa]	fcd [MPa]	fctd [MPa]	fctm [MPa]	Poisson	AlfaT [1/C°]	P.S. [kN/m³]
1	C20/25	25	29960	20	11.33	1.03	2.21	0.2	.00001	25
2	C25/30	30	31470	25	14.16	1.19	2.56	0.2	.00001	25
3	C28/35	35	32300	28	15.86	1.28	2.76	0.2	.00001	25
4	C40/50	50	35220	40	19.83	1.49	3.2	0.2	.00001	25
5										
6										
7										

STEELS FOR BARS

N°	Type of steel	Es [MPa]	fyk [MPa]	fyd [MPa]	ftk [MPa]	ftd [MPa]	ep_tk	epd_ult	ß1*ß2 initial	ß1*ß2 final
1	B450C	200000	450	391.3	540	391.3	.075	.0675	1	0.5
2	B450C*	200000	450	391.3	540	450	.05	.04	1	0.5
3	S235H	210000	235	204.35	360	204.35	0.05	0.04	1	0.5
4	S275H	210000	275	239.13	430	239.13	0.05	0.04	1	0.5
5	S355H	210000	355	308.7	510	308.7	0.05	0.04	1	0.5
6	C1860	200205	1600	1116	1860	1116	0.05	0.04	1	0.5
7										

Set as default   OK   Cancel   ?

Window for the management of the structural materials

Conglomerates Data

- **Concrete Class**
  The conglomerate class must have a standard designation included amongst the indicated ones: C20/25; C25/30; C28/35; C35/45 etc., defined on the basis of the characteristic cylinder compressive strength fck and Rck compression resistance to crushing of sample cubes (expressed in MPa).

- **fck,** cubes [MPa]: This is the characteristic compression strength Rck on cubes to refer to in the course of the project.
- **Ec** [MPa]: This is an elastic module of the concrete to be used in the course of the planning Ec = 220000 [(fck+8)/10]0.3
- **fck** [MPa]: This is the characteristic cylindrical compression strength which on the other hand appears as the first term in the standard designation of the class. In order to pass from the cubic to the cylindrical strength, the expression to be used is fck = 0.83 · Rck
- **fcd** [MPa]: This is the calculation strength of the conglomerate corresponding to acc ·fck / c where acc is the reduction coefficient for long-lasting strengths and is normally 0.85 and c = 1.50 is the partial coefficient of concrete. For level elements (slabs, walls, etc.) with thicknesses lower than 5 cm and realized in place the fcd value must be reduced by 20%.
- **fctd** [MPa]: This is the calculation tensile strength equal to: fctk / c = 0.7 fctm / c
- **fctm** [MPa]: This is the average tensile strength equal to 0.3fck2/3
- **Poisson:** The value of the Poisson ration can vary from 0 (cracked concrete) to 0.2 (non-cracked concrete). The programme uses this coefficient for the calculation of the tangential modulus of elasticity G = 0.50 · Ecm (1+m)
- **AlfaT** [1/C°]: Thermal expansion coefficient.
- **P.S.**[KN/m³]: Unit weight of reinforced concrete. It is used by the programme for the calculation of the weights peculiar to the structural elements.

Steels bars data

- Steel Type for reinforced concrete structures, a single steel type can be used.
- Es [MPa]: Modulus of elasticity of steel.
- fyk [MPa]: Characteristic yield stress assumed in the course of the project corresponding to the rated one 450 N/mm².
- fyd [MPa]: Calculation yield stress assumed in the course of the project corresponding to fyk / s where s is the partial coefficient of steel.
- ftk [MPa]: Characteristic breaking stress assumed in the course of the project corresponding to the rated one (540 MPa).
- Ftd [MPa]: Calculation breaking stress assumed in the course of the project. It can be assumed as equal to fyd (null work-hardening) or to fyd·k where k = ft / fy. This ratio cannot be lower than 1.15 or higher than 1.35. Prudentially, k = 1.15 can be assumed.
- ep_tk: Characteristic unit strain at failure. Its value cannot be lower than 0.075.
- epd_ult: Ultimate calculation deformation corresponding to eud = 0.9 euk
- 1· 2 init.: Steel-concrete adherence coefficient on the first application of the load. It is used by the software in the checking of the cracks in the infrequent operating combinations (SLE, limit operativeness state).
- 1· 2 init.: Steel-concrete adherence coefficient for long-lasting loads. It is used by the program in the checking of the opening of the cracks in the frequent and quasi-permanent operating combination (SLE). Limit operativeness states parameters (Opening of cracks – Normal strains).

Limit operativeness states parameters (Opening of cracks – Normal strains).

- Opening of cracks: In this column are reported the limit values relevant to the opening of cracks fixed according to the limit state and to the fixed environmental conditions (these latter must be indicated in the General Data window).
- S.Cls [aliq. fck]: Limit strain of the operating concrete expressed as a rate of the characteristic breaking stress of concrete.
- S.Fe [aliq. fyk]:Limit strain of the operating steel expressed as a rate of the characteristic breaking stress of steel.

1.4.2 Sections archive

In this archive are defined the sections that constitute the structure of the bulkhead (piles or diaphragms). The environment through which it is possible to mange the sections archive is the following:

Environment for the management of the sections

Through this window you can make many operations on sections. You can add or delete sections (using the "+" and "-" in the central upper part of the window). For the correct definition of a section you must enter the following data:

Section: defines the type of the section to add to the archive. You can choose from the following options:

> o Circular bars - circular cross section with radially spread reinforcements consisting of classic steel bars.
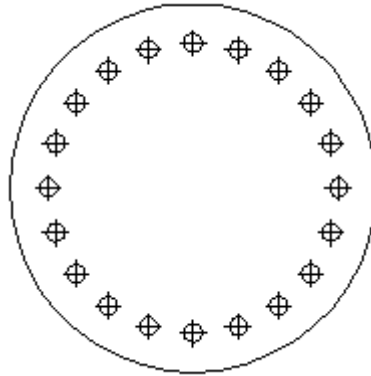
Model of the section Circular bars

For this category of sections the data to be inserted, in addition to data related reinforcements, managed in the Reinforcements options, is as follows:

· Name - name of the section
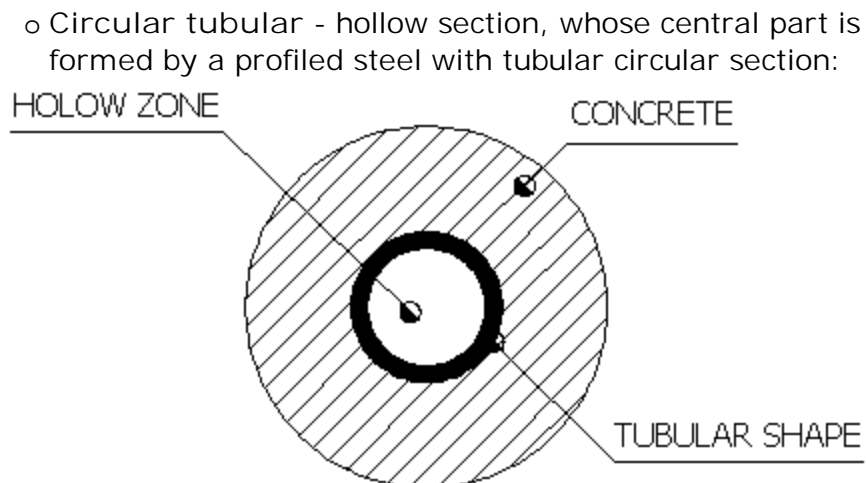· Concrete - select from the materials in Materials archive
· Steel - select from the materials in Materials archive
· Diameter - expressed in [m]
· Arrangement (single row, double row, quincunx)
· Interaxis in 2 directions (the second direction is required only if the arrangement is quincunx) - expressed in [m]

o Circular tubular - hollow section, whose central part is formed by a profiled steel with tubular circular section:

Model of the section Circular tubular

For this category of sections the data to be inserted, in addition to data related reinforcements, managed in the Reinforcements options, is as follows:

· Name - name of the section
· Concrete - select from the materials in Materials archive
· Steel - select from the materials in Materials archive
· Diameter - expressed in [m]
· Arrangement (single row, double row, quincunx)

· Interaxis in 2 directions (the second direction is required only if the arrangement is quincunx) - expressed in [m]
· Reinforcement: details regarding steel profile (diameter, thickness of the tube - expressed in mm)

     o Circular HE shape - hollow section, whose central part is formed by a profiled steel of HE type:

Model of section Circular HE shape

For this category of sections the data to be inserted, in addition to data related reinforcements, managed in the Reinforcements options, is as follows:
· Name - name of the section
· Concrete - select from the materials in Materials archive
· Steel - select from the materials in Materials archive
· Diameter - expressed in [m]
· Arrangement (single row, double row, quincunx)
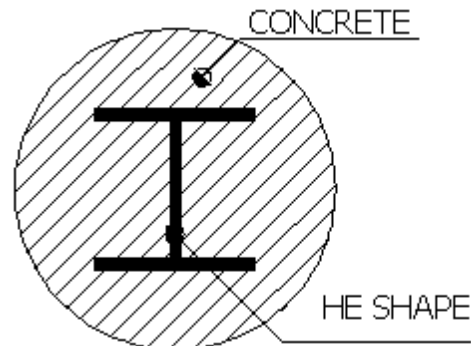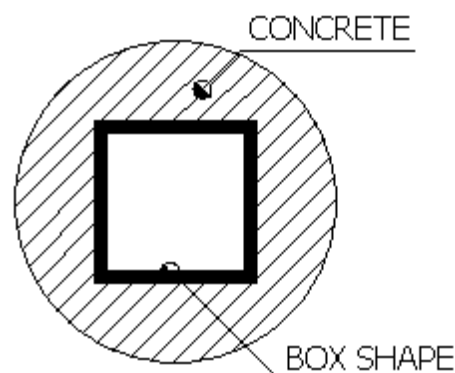· Interaxis in 2 directions (the second direction is required only if the arrangement is quincunx) - expressed in [m]
· Reinforcement: details regarding steel profile (base, height, thickness of the central part (Sa), thickness of wings (Se) - in [mm])

     o Circular box shape - hollow section, whose central part is formed by a profiled steel having a rectangular box section:

Model of section Circular box shape

For this category of sections the data to be inserted, in addition to data related reinforcements, managed in the Reinforcements options, is as follows:

· Name - name of the section
· Concrete - select from the materials in Materials archive
· Steel - select from the materials in Materials archive
· Diameter - expressed in [m]
· Arrangement (single row, double row, quincunx)
· Interaxis in 2 directions (the second direction is required only if the arrangement is quincunx) - expressed in [m]
· Reinforcement: details regarding steel profile (base, height, thickness - in [mm])

   o Rectangular - rectangular section in reinforced concrete:

Model of section Rectangular

For this category of sections the data to be inserted, in addition to data related reinforcements, managed in the Reinforcements options, is as follows:

· Name - name of the section
· Concrete - select from the materials in Materials archive
· Steel - select from the materials in Materials archive
· Arrangement (single row, double row, quincunx)
· Interaxis in 2 directions (the second direction is required only if the arrangement is quincunx) - expressed in [m]
· Section base (Bx)
· Section height (Hz)

   o Input A, E, I, G - generic section for which must be inserted:
· Description
· Area
· Moment of inertia
· Modulus of elasticity
· Shear modulus

For all types of sections is necessary to define the alignment of the reference system. This is the location that the general section should take in the context of structural bulkhead.

For all sections (excluding the generic one) will be calculated the area, moment of inertia and modulus of elasticity.

N.B. The reference system of coordinates is such that the z-axis coincides with the vertical direction, the x-axis with the horizontal axis in drawing plan and y-axis orthogonal to both. From this derives the name Bx and Hz for the geometric data of the rectangular section.
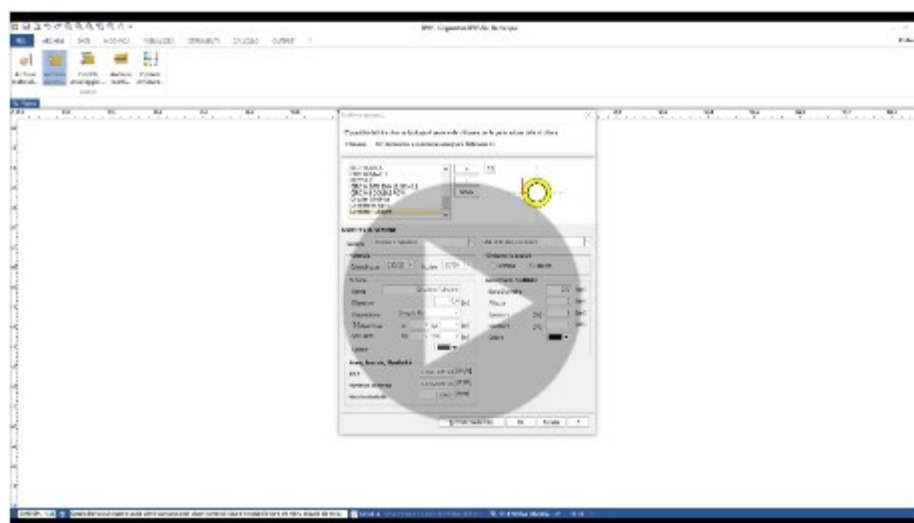
1.4.3   Anchors archive

The anchors are necessary works to supplement the resources of geotechnical and structural strength of the bulkhead. The software allows the management of an anchors archive. A schematic representation, but significant for the quantities characterizing a tie rod anchor is shown in the following figure:
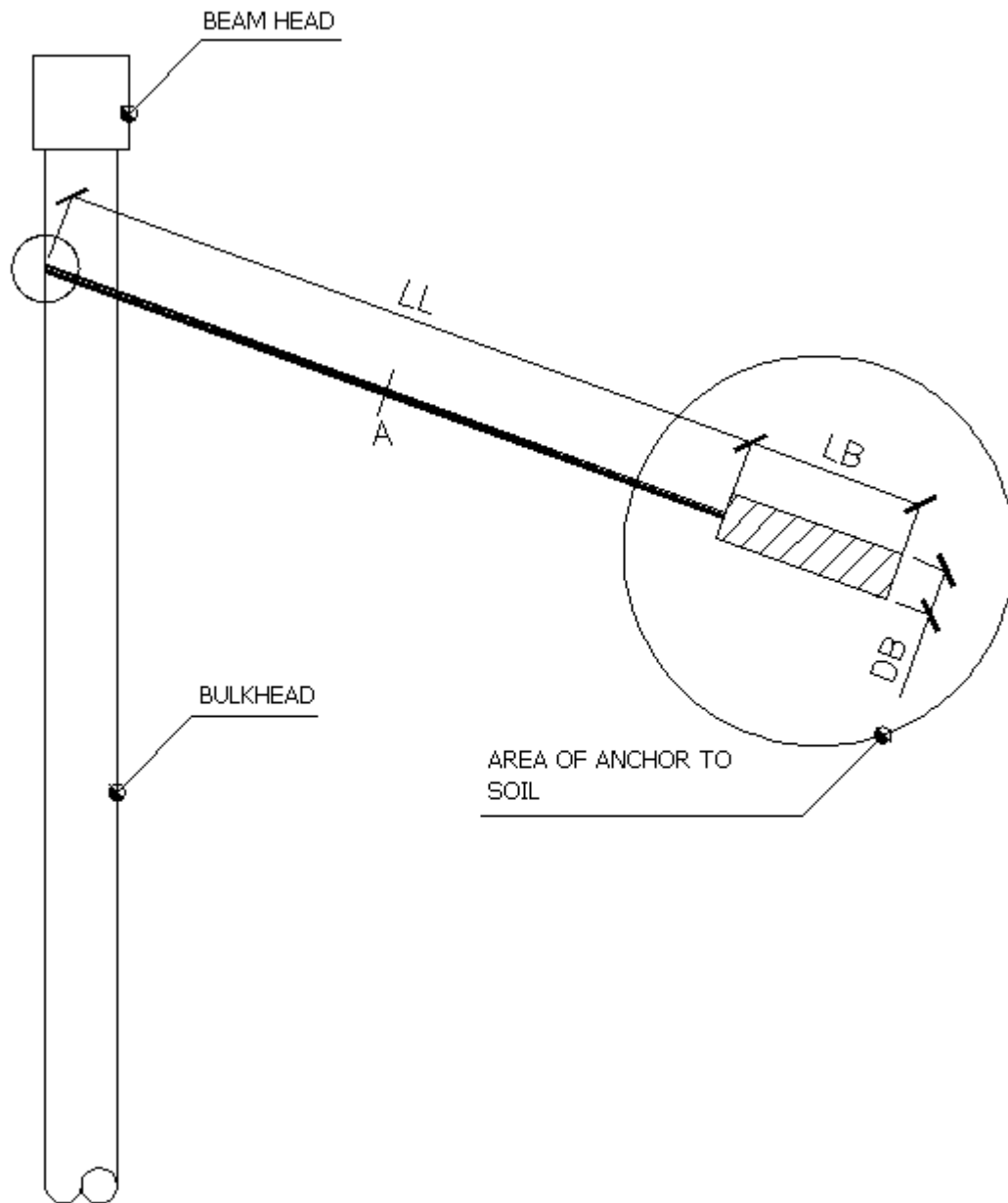
In the image above are presented the following symbols:

- Anchor free length (LL)
- Bulb length (LB)
- Bulb diameter (DB)
- Anchor cross section area (A)

Besides, the zones where the anchor is fastened to the bulkhead (on the upper left side) and the zones where the anchor is fastened to the ground can be recognized as well. Taking into account the previous figure, the environment which allows managing the anchors archive is the following:
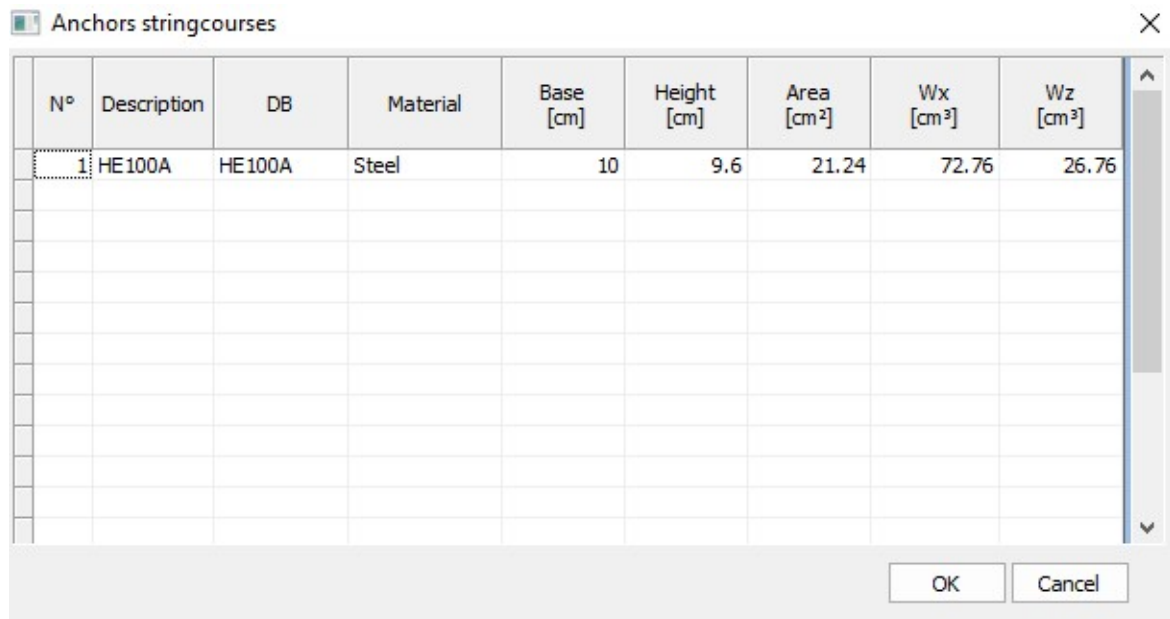
As it can be observed, the data to be entered in order to correctly characterize an anchor are the following:

- **Nr**: identifies the number of the anchor in the archive (ascending order number)
- **Description**: the name through which the anchor is identified inside the archive
- **Area reinforcement**: area of the anchor section which reacts to tensile stress (steel part of the anchor), expressed in cm² (Order of magnitude = 15/20 cm²)
- **Borehole diameter**: the diameter of the borehole made for the insertion of the anchor, expressed in m (Order of magnitude = 0.2/0.8 m)
- **Bulb diameter**: bulb diameter in m
- **Free length**: the length of the anchor reacting to tensile stress (steel part of the anchor), expressed in m (Order of magnitude = 10 m)
- **Bulb length**: the length of the anchor part which reacts by friction with the ground (upon which rely the strength resources connected to friction and adhesion with the ground). With regard to the previous figure, it is the final part of the anchor. It is expressed in m.
- **Materials steel/concrete**: materials to be associated to the type of anchor (from Materials archive)
- **Color**: insert the color which identifies the anchor in the model

NB: for geotechnical and structural checks on the anchor, please refer to Theoretical notes (Anchors limit load).

1.4.3.1   Anchorage stringcourses

Are part of this archive data related to the anchorage stringcourses. The software allows managing an archive for the anchorage stringcourses.

In the window presented above the user can insert new type of stringcourses, alter or delete the existing ones. Data that define an anchorage stringcourse:

- Nr. - order number - delete this to delete the entry
- Description - identifying name
- DB - type of stringcourse to associate from the Database
- Material - constitution material (to choose between concrete and steel)
- Base - width of the beam expressed (Order of magnitude = 10/15 cm)
- Height - height of the beam expressed (Order of magnitude = 10/15 cm)
- Area - section area of the stringcourse
- Wx - section modulus of the section about the reference axis x
- Wy - section modulus of the section about the reference axis y

N.B. The last set of data, namely all those that characterize the geometry of the section (Base, Height, Area, Wx, Wy) are calculated automatically by the program, once you have chosen the stringcourse from the Database. However, you can customize these variables by simply modifying the respective fields of the table.

1.4.4   Reinforcements options

The options relevant to the reinforcements refer to the bulkhead structural planning.

Piles

These data are used for the design and structural checks carried out on bulkheads made up of reinforced concrete piles:

- Diameter of longitudinal bars: diameter of the tubular rods expressed in mm (Order of magnitude = 12/26 mm)
- Minimum number of bars: minimum number to consider
- Longitudinal bars hooks length: expressed in cm (Order of magnitude = 50/150 cm)
- Maximum length of longitudinal bars: expressed in cm (Order of magnitude = 800/1200 cm)
- Clear cover: expressed in cm (Order of magnitude = 4/6 cm)
- Stirrups diameter: expressed in mm (Order of magnitude = 8/10 mm)
- Diameter/No. of stiffening circles: diameter expressed in mm and number of stiffening circles
- Minimum stirrups spacing: expressed in cm (imposed by normative, order of magnitude =15/25 cm)
- [Tubulars] Pile segment anchored to the top: expressed in cm (smaller that the beam head)
- [Tubulars] diameter of the U-bolt bars: expressed in mm

Diaphragms

Data used for the design and structural checks carried out on bulkheads made up of reinforced concrete diaphragms:

- Diameter of longitudinal bars: expressed in mm (Order of magnitude = 12/26 mm), it represents the diameter of the vertical reinforcement
- Wall bars diameter: expressed in mm (Order of magnitude = 10/14 mm), it represents the diameter of the horizontal reinforcement
- Tension/compressive reinforcement ratio: dimensionless number (Usually imposed by the regulations on the basis of considerations also made with regard to the ductility of the section. In any case it will be lower than or equal to 1)
- Minimum reinf. net spacing: minimum net distance between the bars, expressed in cm (must be compatible with the size of the inert material used for the packaging of the concrete; in any case it has an order of magnitude = 2.5/5 cm)
- Maximum reinf. net spacing: expressed in cm (Order of magnitude = 4/8 cm)
- Side clear cover of the bars barycenter: measured starting from the barycentre of the bars, expressed in cm (Order of magnitude = 4/6 cm)
- Stirrups diameter: expressed in mm (Order of magnitude = 8/10 mm), it represents the diameter of the transversal reinforcement
- Minimum stirrups spacing: expressed in cm (Usually imposed by the regulations, in any case Order of magnitude = 15/25 cm)

- Maximum stirrups legs distance: expressed in cm (Order of magnitude 14/26 cm)

Capping beam

These data are used for the planning and structural checks carried out on the head connecting beam realized on bulkheads made up of piles:

- Diameter of longitudinal bars: Expressed in mm (Order of magnitude = 12/26 mm)
- Wall bars diameter: Expressed in mm (Order of magnitude = 10/14 mm)
- Tension/compressive reinforcement ratio: This is the dimensionless number (usually imposed by the regulations on the basis of considerations also made with regard to the ductility of the section. In any case it will lower than or equal to 1)
- Minimum reinf. net spacing: Expressed in cm (must be compatible with the size of the inert material used for the packaging of the concrete. In any case, Order of magnitude = 2.5/5 cm)
- Maximum reinf. net spacing: Expressed in cm (Order of magnitude = 4/8 cm)
- Side clear cover: Measured starting from the barycentre of the bars, expressed in cm (Order of magnitude = 4/6 cm)
- Stirrups diameter: Expressed in mm (Order of magnitude = 8/10 mm)

- Minimum stirrups spacing: Expressed in cm (Usually imposed by the regulations. In any case, Order of magnitude = 15/25 cm)
- Maximum stirrups legs distance: Expressed in cm (Order of magnitude 14/26 cm);

1.5   Data analysis

1.5.1   General data

The general data is equivalent to that seen in the section relevant to "New" menu. The environment for the management of the general data is the following:

We remind, just for the sake of completeness, the general data to be entered.

Project

Input of a synthetic description of the project: description, location, designer and date. It includes the possibility to print this data in the computation report.

Standards

Select the standards regarding the geotechnical checks (GEO standards) or the structural checks (STR standards). For each category of checks, the following choices are possible:

- GEO standards: Eurocode 7 and Eurocode 8
- STR standards: Eurocode 2

Calculation of pressures

In this group of data are defined the theories which can be used for the calculation of the active thrust coefficient, for the passive thrust coefficient and for the limit state coefficient (active or passive) in dynamic conditions. In particular, the following options are available:

- Active pressures: Coulomb, Muller-Breslau, Caqout-Kerisel, Jaky, Rankine
- Passive pressures: Coulomb, Muller-Breslau, Caqout-Kerisel
- Seismic pressures: Mononobe-Okabe, structures that don't undergo displacements
- Application point of the seismic action to 2/3 H, 1/3 H or 1/2 H.

Computation model

It is possible to select a priori the calculation approach for the determination of the stresses and displacements. In particular, it is possible to choose the LEM method (Limit Equilibrium Method) or the FEM method (Finite Elements Method).

Vertical bearing capacity

Is given the possibility to calculate the vertical bearing capacity (Yes/No) or to indicate the correlation factor of the verified verticals referred to the bearing capacity of the pile.

Longitudinal extension

Indicate the longitudinal extension of the bulkhead.

Geometry

In this group of data are set the data sizes with which to initialize the model. In particular:

- Excavation height: it is expressed in m and represents the part of the bulkhead that remains above ground of after the excavation

- Downstream/Upstream ground inclination: in degrees, they affect the value of active and passive thrust coefficients
- Cross section: choose from among the sections defined in the Sections archive. In this first phase of the project, the chosen section is extended to the whole height of the excavation, but further it is possible to differentiate various sections throughout the height of excavation, assigning different types of section.

- Project: Identifies a synthetic description of the project to be executed. It includes the possibility to print this data in course of exportation in rtf format. To do this you must only tick off the square (with a red edge and a diagonal line) situated on the right side of the text box containing the description of the project.
- Date: This is the date which will appear in the calculation report.
- Regulations: It is possible to select the regulations connected to the geotechnical checks (GEO Regulations) or the ones connected to the structural checks (STRU Regulations). For each category of check, the following choices are possible:

  - GEO Regulations: Eurocode 8
  - STRU Regulations: Eurocode

- Calculation of pressures: In this group of data are defined the theories which can be used for the calculation of the active thrust coefficient, for the passive thrust coefficient and for the limit state coefficient (active or passive) in dynamic conditions. In particular, the following options are possible:

Active pressures: It is possible to use the Coulomb's theory, Muller-Breslau's theory or Caquot-Kérisel's theory;
Passive pressures: It is possible to use the Coulomb's theory, Muller-Breslau's theory or Caquot-Kérisel's theory;
Seismic pressures: It is possible to use the Mononobe-Okabe's theory.

- Calculation model: It is possible to select a priori the calculation approach for the determination of the stresses and displacements. In particular, it is possible to choose the LEM method (Limit Equilibrium Method) or the FEM method (Finite Elements Method).

NB: At this level of the programme, it is not possible to change the data relevant to the geometry of the model, since they do not fall within the ambit of the general data of the problem.

1.5.2   L.E.M. - Limit equilibrium method

By selecting the L.E.M. icon, the limit equilibrium method will be used for the analysis of the bulkhead.

The limit equilibrium method is the usual one used for the design practice and is mainly used for the determination of the limit embedment depth. The L.E.M. method is used for works in which it is easily identified the failure mechanism, for example in the presence of cantilevered bulkheads or with a single row of anchors.

For the calculation we consider that the bulkhead is subject to the active thrust upstream and to the passive thrust downstream. The distribution of pressure on the structure is different for bulkheads in cohesionless soils and bulkheads in cohesive soils; also the distribution of thrusts in clayey soil varies in time. The calculation of the thrusts is performed using suitable values of the angle of shearing resistance, unit weight and cohesion, making reference to the thrust coefficients determined according to the classical theories present in the literature (Coulomb, Muller-Breslau, Caquot- Kerisel).

In plotting the pressures diagram are taken into account increases due to: earthquake, groundwater, loads on the embankment. In the computation of passive thrust is introduced a coefficient of safety on passive resistance. For the calculation of the embedment depths it must be proceeded as follows:

(a) Calculation of active and passive thrust coefficients
(b) Assume an initial embedment depth between 0.2H and 0.7H
(c) Calculation of the pressures acting on the work
(d) Equilibrium of moments at the foot (cantilever bulkhead)

The steps (a)-(b)-(c)-(d) will be repeated by increasing the embedment depth to obtain the equilibrium of moments, to which will correspond the sought embedment depth. To remedy the non-equilibrium of the horizontal forces, that depth will be increased by 20%. In case of anchoring, the following mechanisms can be present:

(I) The base of the bulkhead is free to rotate (free support method)
(II) The base of the bulkhead can not rotate (fixed support method)

Free support method (see. Anchors)
To perform the computation, proceed through the steps (a) - (b) - (d). The step (c) will be replaced by equilibrium of moments with respect to the

anchors application point, in this case is not necessary to increase the embedment depth as the equilibrium of horizontal forces is verified.

Fixed support method - Equivalent beam method (see. Anchors)
It is hypothesized that the bulkhead is deformed with reversal of curvature, in this case the problem is not statically determined unless the position of the reversal point is known. If it is assumed that on the inversion point there is a hinge capable of transferring only shear stress (for the purposes of static support), it is possible to break the sheet pile into two equivalent beams. To fix the position of the inflection point, Blum recommends values as a function of flexibility, geotechnical characteristics, etc. Found the location of the inflection point, proceed as follows:
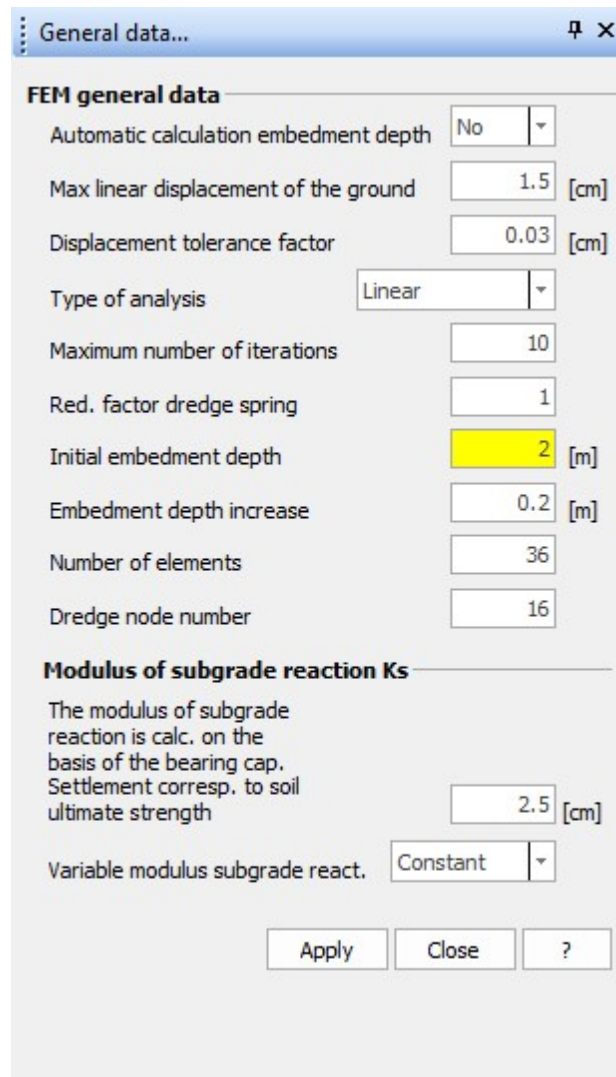
(a) From the equilibrium of moments with respect to the anchors, considering the upper beam at the center of rotation, is determined the reaction of the cart.
(b) From the equilibrium of moments with respect to the foot, considering the lower beam with respect to the center of rotation, is determined the embedment depth.
(c) That depth will be increased by 20%.

1.5.3   F.E.M. - Finite element method

According to the finite element method, the soil is schematized with springs whose characteristics depend on the elastic modules of the soil, differentiating those in compression by those in tension. Bowles proposed to calculate, in an approximate way, the value of Ks *(reaction module that is connected to the stiffness of the soil)* on the basis of the bearing capacity of the foundations. The method provides directly, after building the global stiffness matrix and the vector of nodal loads, the deformations and, from these, the moments and the nodal reactions. For the calculation of the bulkhead proceed as follows:

(a) Calculation of the side pressure to the dredge line
(b) Set a depth for the first attempt
(c) Estimation of the value of Ks below the dredge line
(d) Arrangement of nodes where are assigned the stiffness of the springs
(e) Establish a trial section and calculation of the inertia moment of the section
(f) Stiffness calculation of any anchors
(g) Assemble the global stiffness matrix
(h) Assembly of the vector of nodal loads
(i) Calculation of the nodal displacements

The calculation is solved by an iterative procedure. The iterations continue until the displacements on the dredge line, between two cycles of calculation, are included in a tolerance value specified. The environment for the management of certain data related to implementation of the finite element method is the following:

FEM general data
- Automatic calculation of embedment depth: Select Yes for automatic computation of the embedment depth and No to let the input of the embedment depth at the choice of the user.
- Max linear displacement of the ground: Expressed in cm. Is the maximum displacement that allows to consider the ground in the linear range. After this displacement, the spring which represents the soil can not be considered in elastic-linear field (depends very much on the characteristics of the soil, in any case Order of magnitude = 1/2 cm).

- Displacement tolerance factor: Expressed in cm. Is the tolerance set to define the exit condition from analysis iterations (depends on certain analysis conditions and geometry of the structure, Order of magnitude = 1/200 of the maximum size of the structure for example bulkhead height).
- Type of analysis: Defines whether the analysis conducted is linear or non-linear (Is recommended non-linear analysis when the statics of the problem depends predominantly on the geotechnical aspect of the problem).
- Maximum number of iterations: It's the maximum number of iterations to be carried out to find a solution to the movement. Above this limit, the solution is deemed not found (In contexts related to practice the 'order of magnitude iterations = 5/10).
- Red. factor dredge spring: It's a dimensionless factor that goes to multiply the reaction modulus of the spring located at the bottom of the excavation. Must assume the value less than or at most equal to 1.
- Initial embedment depth: Embedment depth for the first trial, expressed in m. (Order of magnitude = 0.1 / 0.2 of the height of the bulkhead).
- Embedment depth increase: Is the increase for the current embedment depth with which to search for a balanced solution (Magnitude = 0.1 / 0.2 of the height of the bulkhead).
- Number of elements: Must be between 10 and 50. Is the number of finite elements in which is discretized the entire bulkhead (it should realize a rational description, not too sparse, to avoid gross errors in the solution, not too thick, to avoid that the calculation times are extended considerably).
- Dredge node number: Defines the node associated with the bottom of the excavation. The higher this number will be, bigger the number of finite elements that discretize the upper part of the bulkhead will be.

Modulus of subgrade reaction Ks
- Settlement corresponding to soil ultimate strength: Is the displacement associated to the bearing capacity, expressed in cm.
- Variable modulus of subgrade reaction: Is possible to take into account the variability of the axial stiffness of the springs, which simulate the presence of the ground, with the depth or even conduct the analysis with a constant modulus.

⚠️
Using the F.E.M. method can be checked an embedment depth assigned by the user (for example, to verify existing bulkheads), by selecting No for the automatic calculation of

the embedment depth and set to 1 the maximum number of iterations (imposed automatically) and assign the desired initial embedment depth.
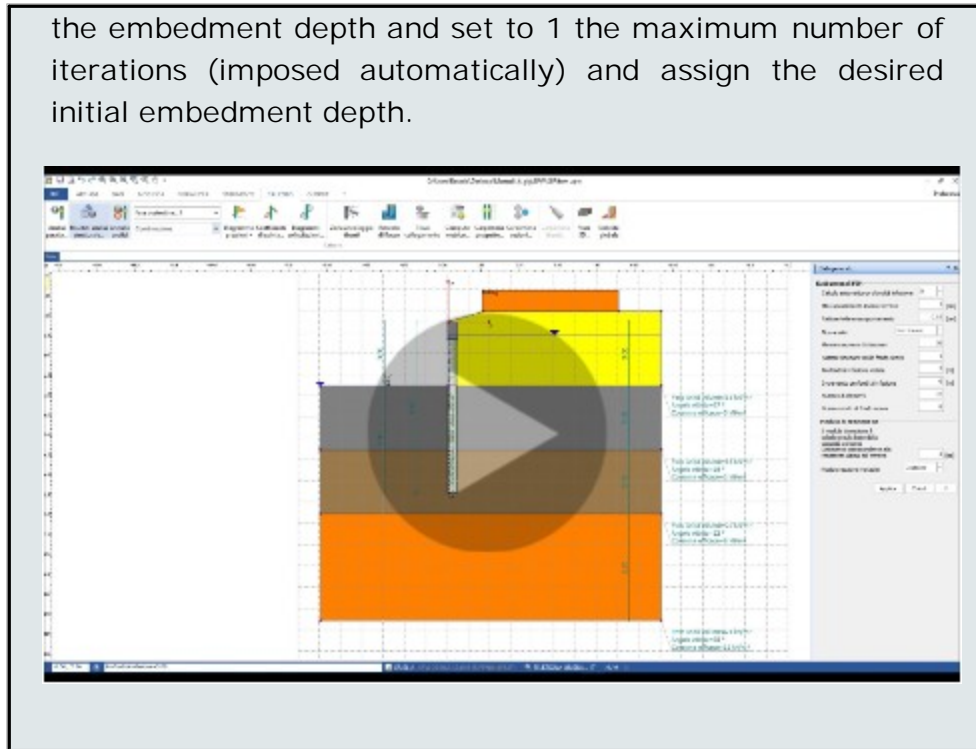
1.5.4 Ground geometry

The data relevant to the ground geometry is necessary for the definition of the topographic profile of the ground. The environment for the management of the ground profile is the following:

Ground geometry 📌 ✕

/**Angles and distances** \ Coordinates \
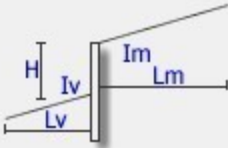
Excavation height	H:	0	[m]
Downstream stretch length	Lv:	0	[m]
Downstream ground inclination	Iv:	0	[°]
Upstream stretch length	Lm:	0	[m]
Upstream ground inclination	Im:	0	[°]

Generate coordinates...

No. Insertion vertex

Gradients for the calculation of the profile

Upstream/downstream Inclination: 0   0 °

The inclination angle of the slope cannot be higher than the ground angle of friction

☐ Numbering of the vertices

Apply	Close	?

General data...

Data entered by coordinates

Enter the coordinates of vertices which define the profile with regard to a fixed reference system. The vertices must be entered from downstream to upstream in terms of x-z coordinates. The coordinates must be expressed in m. The inclination of the upstream and downstream profile must be also entered. It is possible to view the numbers of the vertices.

Data entered by angles and distances

- Excavation height (H): Expressed in m, it represents the part which will remain above ground after the execution of the excavation.
- Downstream stretch length (LV): Expressed in m; it is necessary to carefully estimate this quantity especially for the analysis of the global stability and for the analysis of the filtration phenomenon (Order of magnitude = 1/2 times the excavation height).
- Downstream ground inclination (IV): Expressed in degrees (depends mainly on the topographic conditions of the problem).
- Upstream stretch length (LM): Expressed in m (the same considerations made for the length of the downstream stretch are valid).

- Upstream ground inclination (IM): Expressed in degrees (its value is based on the topographic configuration of the case under examination).
- Gradients for the calculation of the profile: Expressed in degrees, both upstream and downstream, they represent the values used by the software for the calculation of the thrust with inclined profile; their value is based on the topographic configuration of the case under examination, but it must be specified that the formulas used for the calculation of the active and passive thrusts generally have validity limitations precisely on such parameter.
- It is possible to view the numbers of the vertices.

N.B.: The insertion of the data in terms of angles and distances is only an integrative – not a substitute – instrument of the insertion of the vertices in terms of coordinates. In fact, also after the insertion of the angles and distances it is necessary to click on the Generate coordinates button, which lets you go back to the window relevant to the vertices.

Seeing the importance that the insertion of the data in terms of coordinates of the vertices has, some specifications must be made.
- The reference system with regard to which the coordinates of the vertices are defined has always its origin in correspondence of the head of the bulkhead;
- The succession of the vertices must be entered in the downstream-upstream order.

You can refer to the following figure:
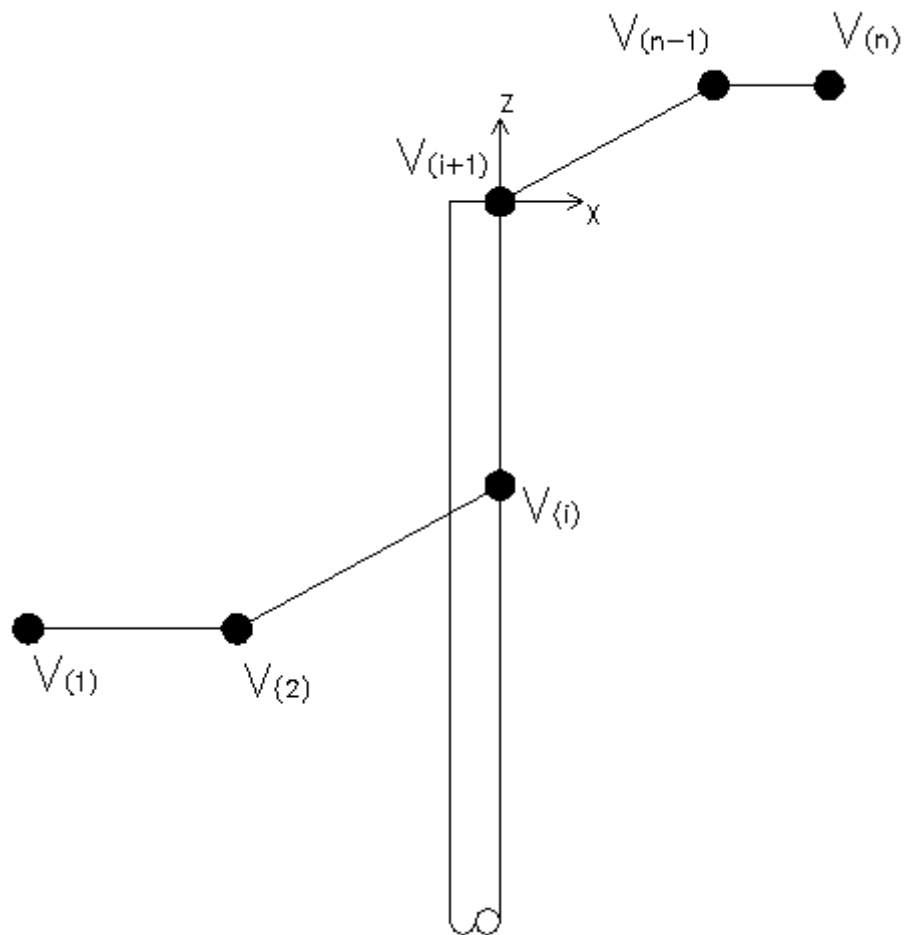
We report, for the sake of completeness, a guide figure for the insertion of the geometrical data by angles and distances.
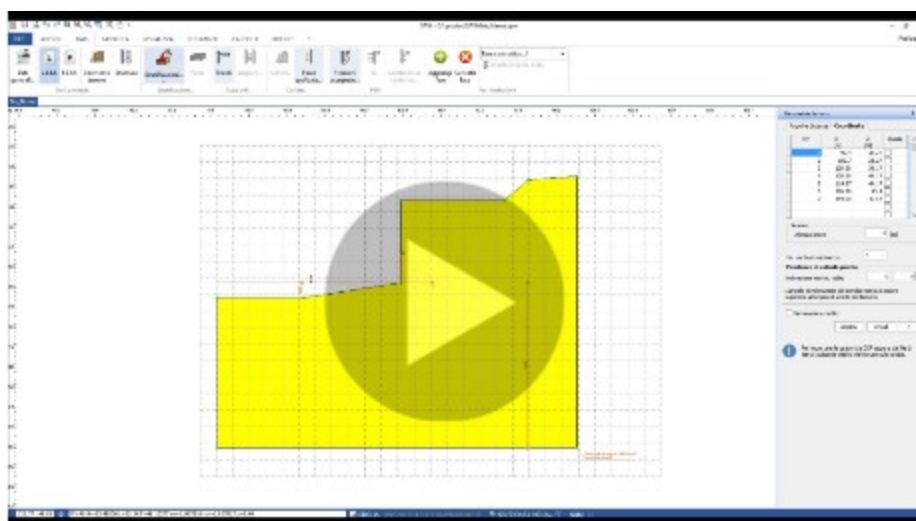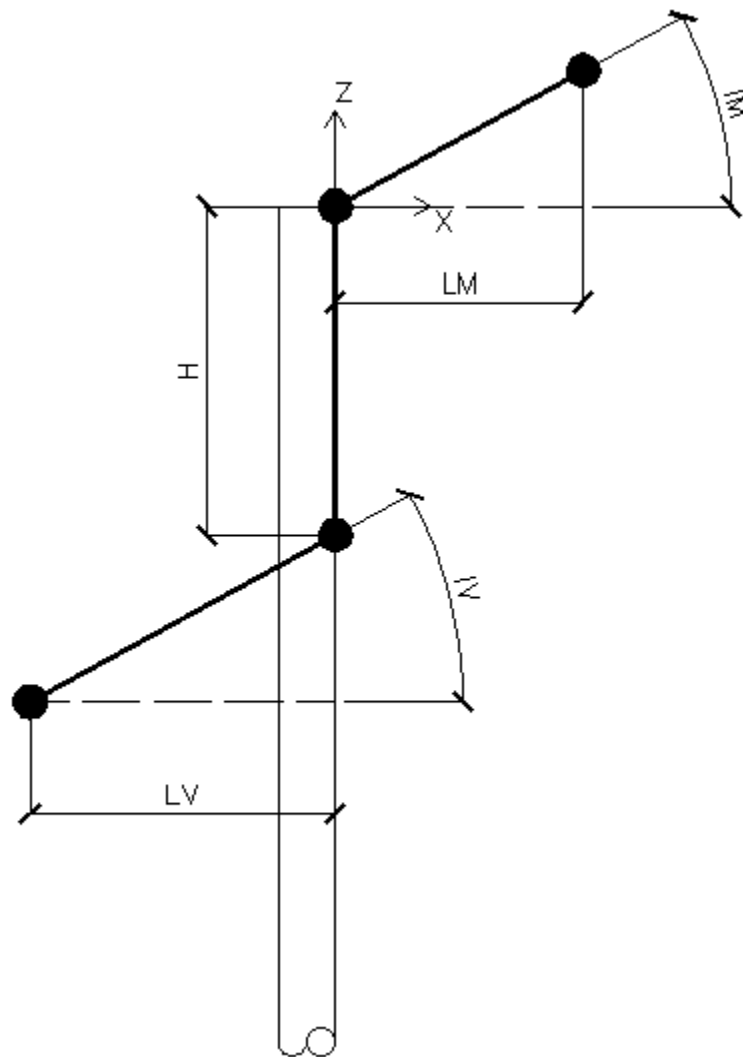
1.5.5 Structure

The data relevant to the structure concern the structural composition of the bulkhead. Through this set of data we then define the section (or the sections, if need) of the bulkhead, materials, etc. The environment for the management of the data relevant to the structure is the following:

Structural composition

The structural composition is obtained by assembling elements that can have different sections. So, for instance, it is possible to use, for the same bulkhead, different resistant sections on the excavation depth. The

following image provides an explanatory description of the previous concept:
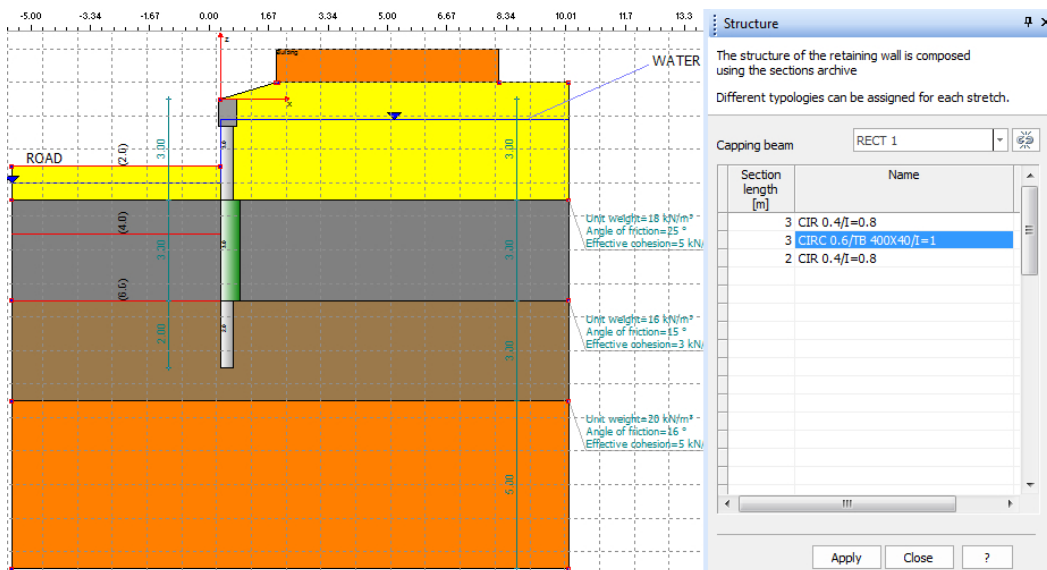
The data to be entered for the structure refer to each part in which the section can be susceptible of changes. For each segment it is therefore necessary to define the following:

- Section length: Expressed in m, it represents the constant-section segment of the excavation height; it is a good practice to use sections with equal features for the whole bulkhead; in any case, where it becomes necessary to use more types of sections, it is advisable to avoid too short segments.

- Name: Type of section to be associated to the considered segment - choose amongst the different types of sections defined in the Sections archive.

N.B.: *The lengths are referred to each section and the structural continuity of the work (from the end of previous section) must be respected. For the first section, the length is defined wit regard to the zero of the reference system.*

- Capping beam: From the drop-down menu Capping beam can be selected the section that represents the capping beam. Clicking the Apply button will insert the capping beam. To remove it click on the button next to the drop-down menu.

1.5.5.1 Stratigraphy

Different soil layers can be defined in the stratigraphy window:
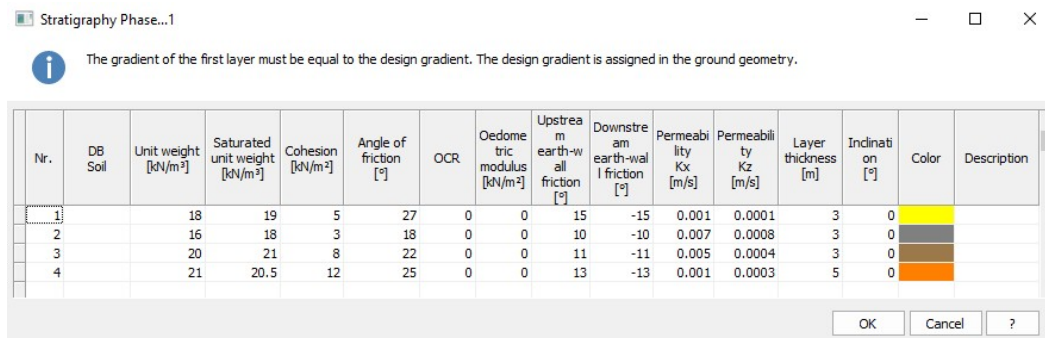
For each layer, the following data must be defined:

- Nr: Identifies the layer by an ascending numeric index from surface in depth.
- DB Soil: Allows setting initial characteristics derivable from a soils database provided with the software. The database can be modified by altering existing characteristics or deleting/adding soil types (in the installation folder, search for Soils/Soils_EN.csv).
- Unit weight: Expressed in kN/m^3 (order of magnitude = 17/20 kN/m^3), it represents the natural weight of the soil.
- Saturated unit weight: Expressed in kN/m^3 (order of magnitude = 18/21 kN/m^3), it represents the saturated weight of soil - to be entered in the presence of groundwater. In the presence of groundwater, for the analysis in drained conditions, the software estimates the effective pressures starting from the effective unit weight.
- Cohesion: Expressed in kN/m^2 (order of magnitude 1/5 kN/m^2).
- Angle of friction: Expressed in degrees (order of magnitude = 22/30°).
- Over-Consolidation Ratio (OCR): Depends on the tensional history of the site under examination (order of magnitude = 1/2)
- Oedometric modulus: Is the normal modulus of elasticity estimated in oedometric conditions, expressed in kN/m^2 (Order of magnitude = 10000 kN/m2).
- Upstream earth-wall friction: Expressed in degrees (order of magnitude = 10/12°). Usually the regulations impose values no higher than 2/3 of the angle of internal friction of the ground.
- Downstream earth-wall friction: Expressed in degrees (order of magnitude = 10/12°). Usually the regulations impose values no higher than 2/3 of the angle of internal friction of the ground.
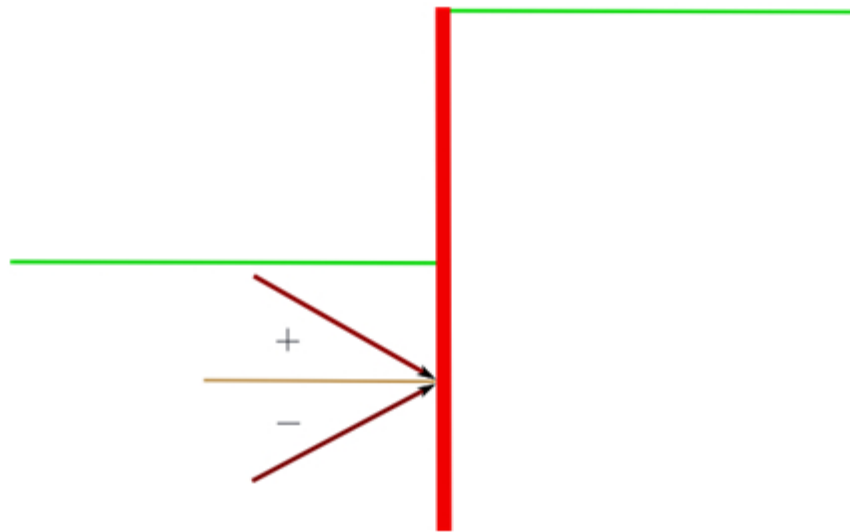
Direction of the passive thrust

The soil-downstream wall friction angle must be less than j/2. Otherwise it is necessary to consider the non-coplanarity of the sliding surfaces.

- Permeability Kx: Expressed in m / s (parameter necessary to plot the flow grid)
- Permeability Kz: Expressed in m / s (parameter necessary to plot the flow grid)
- Layer thickness: Estimated starting from the lowest point of the layer previous to the one to be defined; it is expressed in m.
- Inclination: Expressed in degrees.
- Color: Identifies the layer inside the drawing area.
- Description: Name associated by the user to the inserted layer.

Some clarifications must be made as regards the definition of the layer thickness and inclination.

Layer thickness: The thickness of the layer is measured along the vertical passing through the origin of the fixed reference system (which, bear in mind, coincides with the head of the bulkhead). For more clearness, please refer to the following figure:
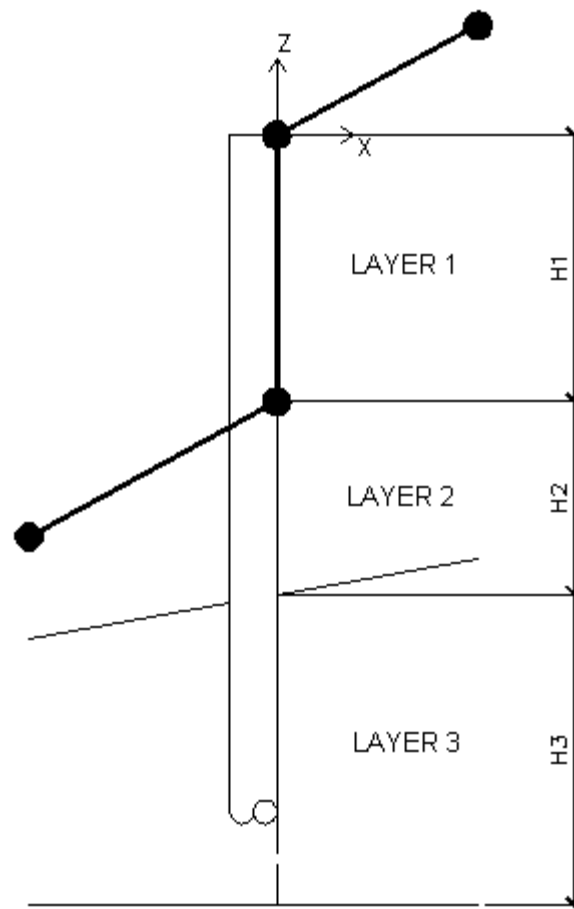
Layer inclination: The inclination of the layer is the angle of rotation relevant to the bottom of the layer taken into consideration. The rotation of the layer is defined with regard to the pole obtained through the intersection between the vertical line passing through the origin and the horizontal line which identifies the lower part of the layer to be defined.

When the embankment is made up of cohesive soil, traction zones can be expected to occur.
You don't have to rely on the traction zone to reduce lateral pressure.
Instead, it must be assumed that it can be formed and filled with water.
In the presence of a traction zone it is suggested to adopt both the indications shown in the figure, to consider the traction block as an overload and to consider the thrust of the water contained in the traction fractures.
It is known that cohesive soils tend to lose cohesion when exposed to excavation due to moisture absorption and/or formation of tensile fractures.
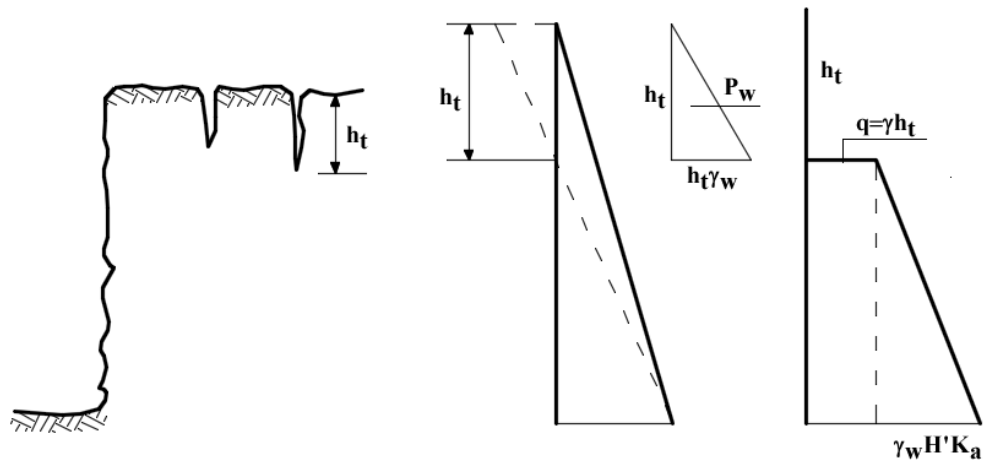
Tensile fractures and pressure diagram suggested in the presence of cohesive soils

1.5.6   Graundwater

The presence of a possible groundwater conditions the calculation both from the geotechnical and structural point of view. That is why in the software the effect of the groundwater is taken into consideration including possible filtration problems.

As regards the groundwater, the data to be entered is the following:

- Upstream groundwater depth: Is the depth – with respect to the horizontal level passing through the reference general system – of the free surface of the groundwater upstream the bulkhead, expressed in m.
- Downstream groundwater depth: Is the depth – with respect to the horizontal level passing through the reference general system – of the free surface of the groundwater downstream the bulkhead, expressed in m.
- Siphoning check: Allows carrying out or not the siphoning check, executed upon the first (or the shortest) flowline.
- Activates presence of groundwater in this phase: As it has already been mentioned above, the software allows defining more calculation phases. Through this information it is possible to choose whether the groundwater must be considered in the calculation of the current phase.
- Thickness impermeable layer: Identifies the depth at which the impermeable layer is situated, expressed in m.
- Flowlines spacing: Defines the display density of the flowlines, expressed in m.
- View Mesh: Allows viewing the calculation grid used for the solution of the filtration problem.
- View Flowlines: Allows the display of the flowlines. It is possible to choose the colors of the calculation grid and flowline.
- Generate flow grid: Allows carrying out the filtration analysis.

As regards the data which defines the profile of the groundwater it is advisable to refer to the following figure:
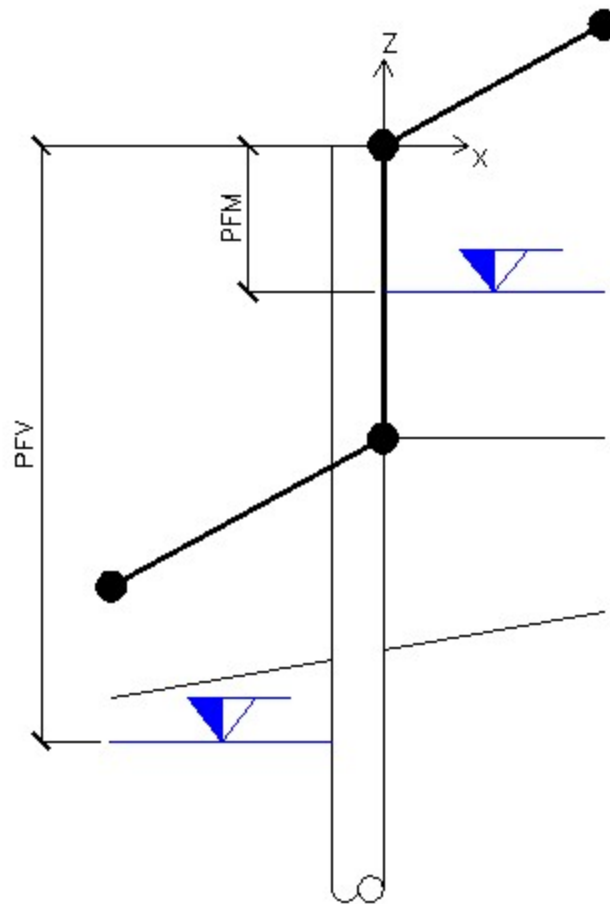
In the figure, PFM is what in the groundwater data window is defined as the upstream groundwater depth, while PFV is what in the groundwater data window is defined as the downstream groundwater depth.
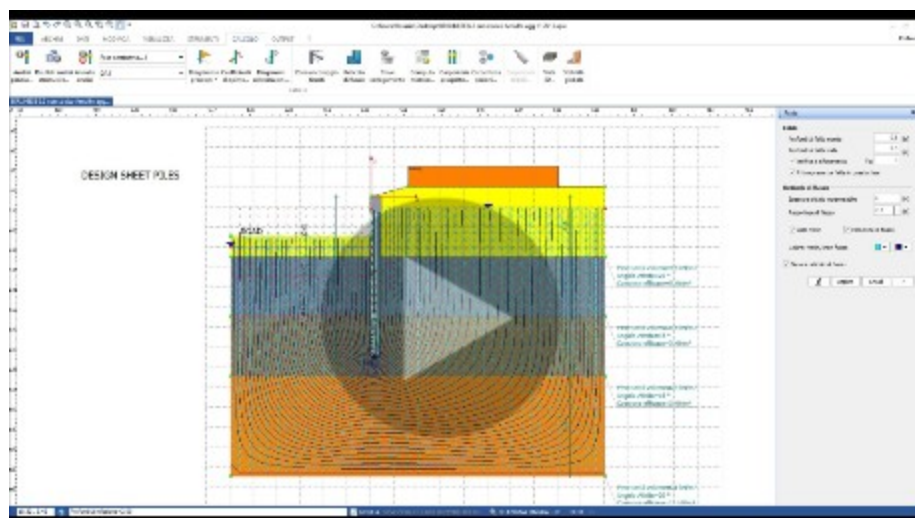
1.5.7   Anchors

This subject has been partially treated in the Anchoring stringcourses section. In this section we are about to discuss the insertion of the anchors.

For a correct definition of the fastening anchors, it is advisable to enter the following data:

- Description: Allows the user to identify – by means of a name – the anchor inserted.
- x: It is the abscissa, measured in the general reference system, where the anchor is inserted. It is expressed in m.
- z: It is the level, measured along the vertical passing through the general reference system, where the anchor is inserted. It is expressed in m.
- Inclination: It is the angle of inclination that the axis of the anchor forms with the horizontal axis of the general reference system. It is expressed in degrees (n.b. the inclination is defined as positive when clockwise).
- Interaxis: It is the distance, measured in the direction perpendicular to the drawing plane (general direction y) existing between two adjoining anchors. It is expressed in m.
- Angle of friction: It is the angle of friction between the anchor bulb and the ground where this latter is fastened. It is expressed in degrees.
- Adhesion: It is a measure of the cohesion which is measured on the interface between the bulb and the soil (in the anchoring area). It is expressed in kN/m².
- Typology: Refers to the type of anchor. It must be chosen from the Anchors archive.

- Stringcourse: It is the stringcourse by means of which the anchor is fastened to the bulkhead. It must be chosen from the stringcourses archive.
- Active/Passive: Is the option through which the user can decide whether the anchor is active (pre-stressed anchor) or passive (unloaded anchor in the initial phase of the structure's life).
- Initial tensile load: In case the anchor is active, it defines the extent of the pre-tension. It is expressed in kN.
- Bearing capacity: It is the bearing capacity of the anchor. If set to zero, it will be calculated automatically by the software.
- Safety factor: Imposed safety factor with regard to the collapse of the anchor.
- Rowe's reduction factor: It is a coefficient which depends on the deformability of the piling. This coefficient reduces the maximum moment calculated on the bulkhead.
- Thrust coefficient: Please refer to the calculation of the anchors bearing capacity.
- Free of fixed end (see L.E.M.): Detects if the insertion of the anchor involves a fixed bound for the bulkhead (Fixed end) or, on the contrary, it can be considered as non-existent in terms of bounds (Free end).

1.5.8 Struts

The supports are elements aimed at increasing the resistance resources of the bulkhead. However, unlike the anchors, they are mostly subject to compression. Consequently, the nature of the checks to be carried out changes. In the following figure is schematically shown the use of a strut support:

Strut support
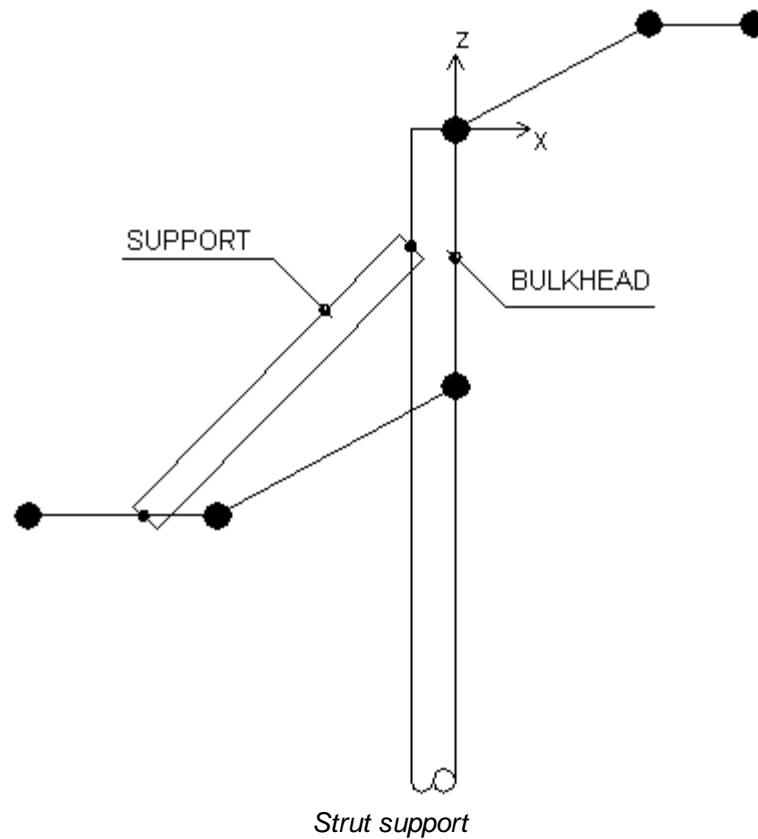
The strength checks carried out on the strut are the ordinary ones executed on compressed members.

- Compression strength test: In this test is estimated the maximum compression stress, comparing it with the breaking stress of the compressed element.
- Check concerning the collapse due to instability: As known, compressed members are subject to the problem of structural instability. It is therefore necessary to check that the normal stress acting upon the strut is lower than or equal to the critical load of the strut.

The strut is an element which can only be inserted if is chosen the F.E.M. method, and the element only reacts if a displacement of the bulkhead is activated in a downstream direction.

Data regarding the struts can be inserted in the following window:

- **Description**: Allows the user to identify – by means of a name – the strut inserted.
- **x**: It is the abscissa, measured in the general reference system, where the strut is inserted. It is expressed in m.
- **z**: It is the level, measured along the vertical passing through the general reference system, where the strut is inserted. It is expressed in m.
- **Length**: Length of the strut, expressed in m.
- **Inclination**: It is the angle of inclination that the axis of the strut forms with the horizontal axis of the general reference system. It is expressed in degrees (n.b. the inclination is defined as positive when clockwise).
- **Area**: Area of the strut's section, expressed in m^2.
- **Interaxis**: It is the distance, measured in the direction perpendicular to the drawing plane (general direction y) existing between two adjoining struts. It is expressed in m.
- **Material**: Material to associate to the strut, can be chosen from Materials archive.
- **Modulus of subgrade reaction**: Strut's modulus of subgrade reaction, equal to the axial stiffness of the element, expressed in kN/m.
- **Color**: Gives the possibility to assign a color to the strut.

1.5.9 Loads

The software offers the possibility to take into consideration the presence of possible distributed loads, considered as Lines, Strips or Uniform Loads.

.oads on the ground. Phase...1   ✕

Description	Type	Xi [m]	Xf [m]	zi [m]	Q [kN]-[kPa]	Color	
Building	Strips	1.6	8	0.5	12		
	Lines						
	Strips						
	Uniform						

Apply   OK   Cancel   ?
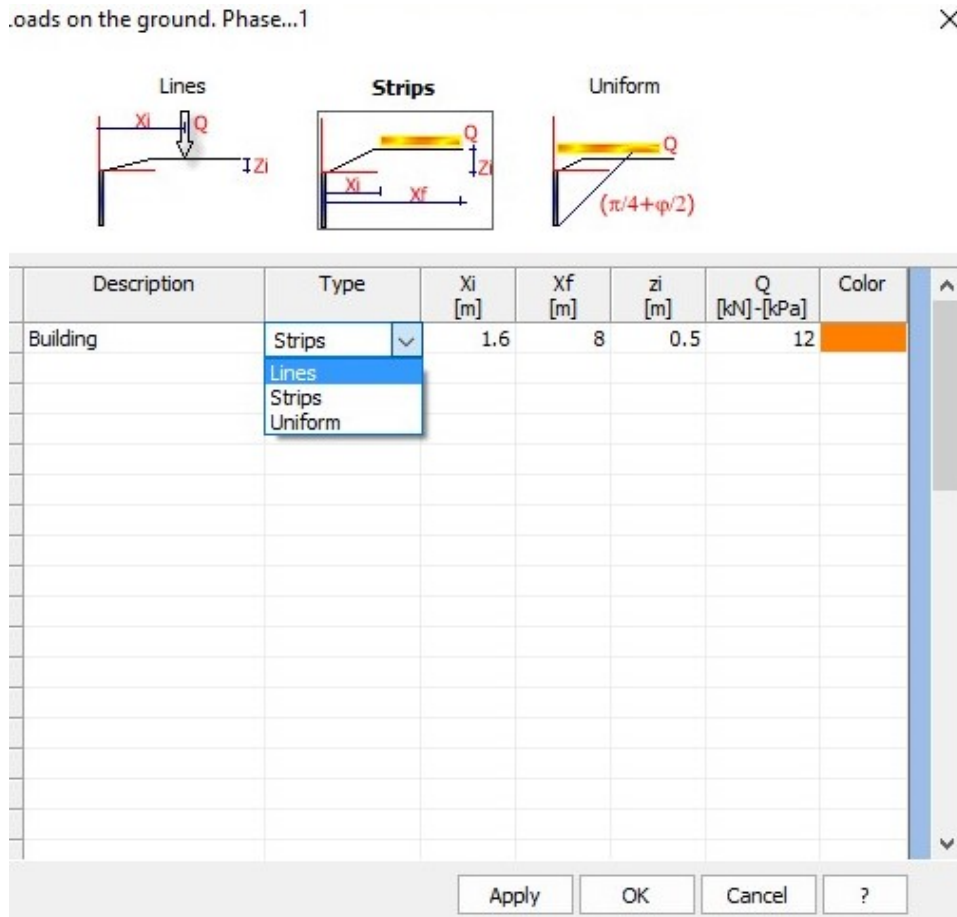
The data to be entered for a correct definition of a load is the following:

- **Description**: Identifies the load; data necessary for the identification of the action in the load combinations.
- **Type**: Choose amongst load lines, load strips and uniformly distributed load.
- **xi**: It is the abscissa starting from which the load starts acting. In case of load lines, it is the abscissa which defines the application of the load line; it is expressed in m.
- **xf**: Data required if load strips or uniform loads are defined. It is the final abscissa of the load (so, it defines the extension of the load); it is expressed in m.
- **zi**: Depth expressed in m, currently only having graphic value, therefore the increase of stress induced by the overload is still evaluated starting from the head of the bulkhead.
- **Q**: Load - expressed in kN (in case of load line) or in kN/m² (in case of strip and uniform load).
- **Color**: Assignation of the color to be used for the display of the load strip.

N.B. The loads are distributed per linear meter in the longitudinal direction.

For the interpretation of the different types of load, it may be useful to have a look at the following figure:
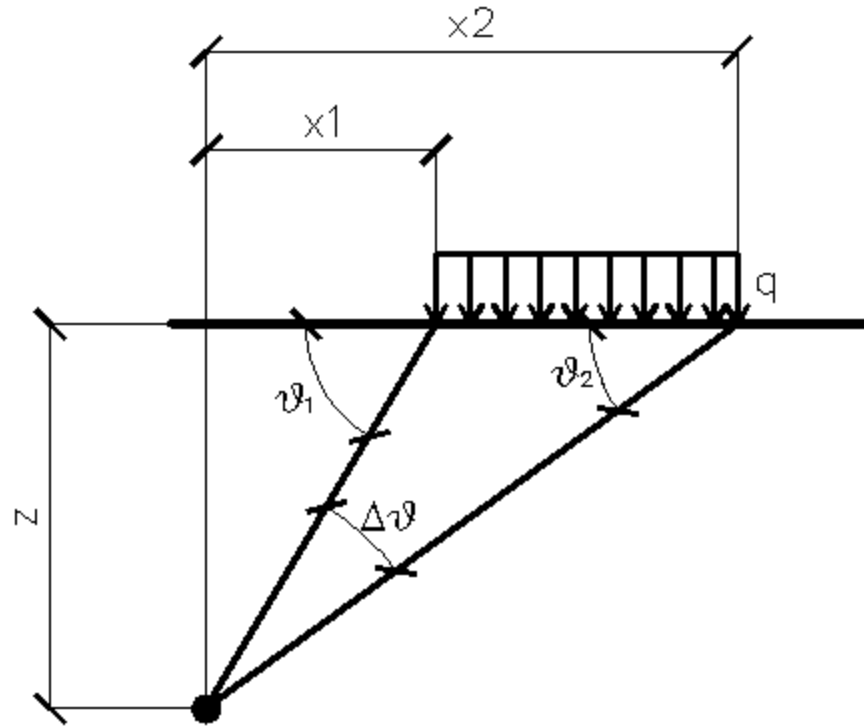
Orthogonal load strips

For load strips, the distribution of the stresses is estimated according to the depth z.

A partially distributed load with an initial abscissa x1 and a final abscissa x2 generates a diagram of pressures upon the wall whose values have been determined according to the TERZAGHI formulation, which expresses the pressure at a generic z depth, as follows:

$$\sigma_q(z) = \frac{Q}{2\pi \times (2\Delta\theta + A)}$$

$\Delta\vartheta = \vartheta_1 - \vartheta_2;$

$A = \sin(2\vartheta_1) - \sin(2\vartheta_2)$

$B = \cos(2\vartheta_1) - \cos(2\vartheta_2)$

$\vartheta_1 = \text{arctg}(z/x1)$

$\vartheta_2 = \text{arctg}(z/x2)$

By integration, the resultant and the relevant arm will be obtained.
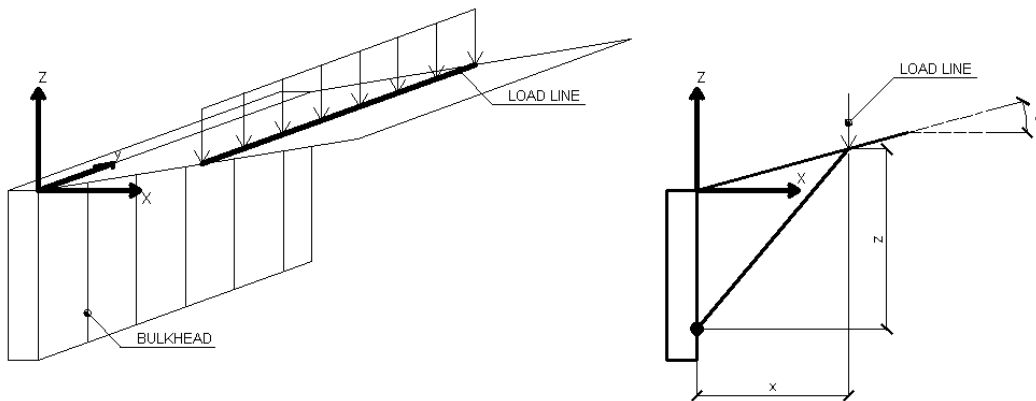
Schematization of the load lines

In this case too, as for the load strips, the distribution of the stresses is estimated according to the depth z. The load lines generate an increase in the pressures upon the wall, which, according to BOUSSINESQ – at the depth z – can be expressed as follows:

$$\sigma_x(x,z) = \frac{2V}{\pi \cdot x^2 \cdot z \cdot (x^2 + z^2)^2}$$

$$\tau_{xz}(x,z) = \frac{2V}{\pi \cdot x \cdot z^2 \cdot (x^2 + z^2)^2}$$

Where the symbols have the following meaning:

V = Intensity of the load expressed in [F/L]

X = Distance, in horizontal projection, of the point of application of the load from the wall

If the action plane is inclined by , the reference system (x,z) is rotated in (X,Z) through the following transformation:

$$\begin{cases} X = x \cdot \cos(\varepsilon) - z \cdot \sin(\varepsilon) \\ Z = z \cdot \cos(\varepsilon) + x \cdot \sin(\varepsilon) \end{cases}$$

A load Q, uniformly distributed on the ground surface induces constant pressures equal to:

By integrating the stress indicated in the above formula, we obtain the total thrust due to overload:

$$S_q = k_a \cdot Q \cdot \frac{\sin(\beta)}{\sin(\beta + \varepsilon)} \cdot H$$

With the point of application to H/2 (being the stress distribution constant). In the above formulas the symbols have the following meaning:

= Inclination of the inner wall to the horizontal plane passing through the foot

= Inclination of the ground surface to the horizontal - positive if counterclockwise

Ka = Coefficient of active pressure calculated in the previous paragraph.

N.B. The uniform load is distributed automatically by the head of the bulkhead across the width of the failure wedge.

1.5.10 Applied forces

The software allows considering *(only for the F.E.M. analysis method)* forces and moments like concentrated loads acting upon the bulkhead.
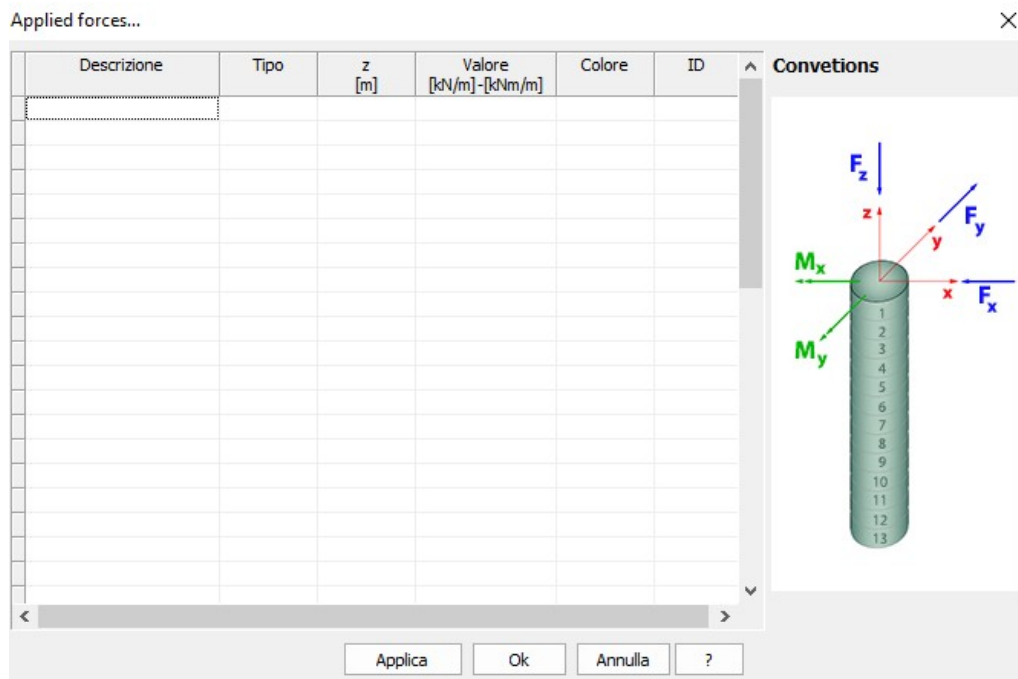
For a correct definition of a concentrated force, it is necessary to define the following data:

- Description: This information identifies the force with a name; its insertion is necessary for the purposes of its identification in the combinations of loads.

- Type: This information identifies the type and direction along which the force acts. To choose amongst the following types of loads:

  o Fz, expressed in kN/m. It is a vertical force, whose direction is thus coincident with the vertical reference axis
  o Fx, expressed in kN/m. It is a horizontal force whose direction is coincident with the horizontal reference axis contained in the drawing plane
  o Fy, expressed in kN/m. It is a horizontal force whose direction is coincident with the reference axis orthogonal to the drawing plane
  o Mx, expressed in kNm/m. It is a moment whose axis vector is the horizontal reference axis contained in the drawing plane
  o My, expressed in kNm/m. It is a moment whose axis vector is the reference axis orthogonal t the drawing plane

- z: Expressed in m. It is the level at which the application point of the force (or moment) is positioned.
- Value: It is the value corresponding to the intensity of the force. The sign agrees with the global reference system; it is expressed in kN/m (for the forces) or in kNm/m (for the moments).
- Color: It is the color used to view the drawing of the force.
- I D: It is an identification index of the force.

1.5.11 Pressures assigned

The distribution of the pressures acting upon the bulkhead *(including when you have to calculate the nodal forces using the FEM method)* is determined on the basis of the application of classical methods *(e.g. calculation of the horizontal pressure using Rankine's method)*. However, it is possible to be confronted by situations in which the distribution of the horizontal pressures, though it is known, does not follow the expected progress. The software allows managing these situations through the manipulation of the calculated distribution of pressures, integrating or replacing it with a distribution inserted by the user. the environment for the management of the pressures assigned is the following:

Pressure diagram assigned Phase:1 ✕

Z [m]	value [kPa]	Add to diagrams	Color	
		☐		^
		☐		
		☐		
		☐		
		☐		
		☐		
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		☐		v

[OK] [Cancel] [?]
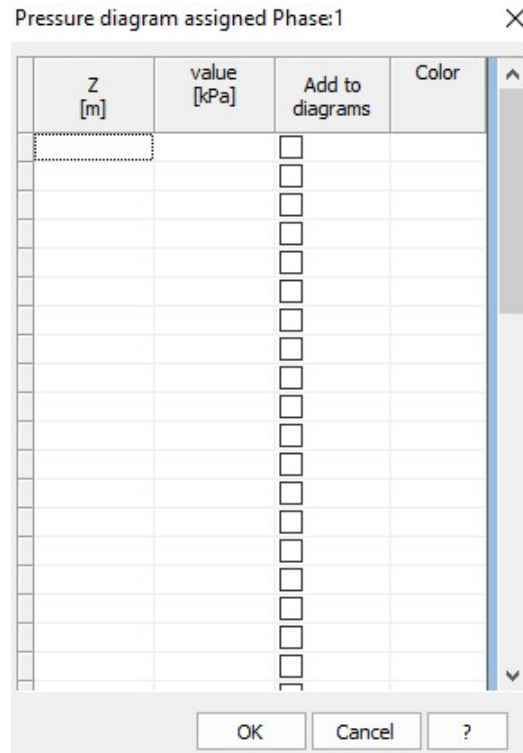
- Z: Expressed in m. It is the elevation to which is required that the the pressure assumes a given value.
- Value: Expressed in kPa. It is the value of the pressure corresponding to the elevation z.
- Add to diagram: This check is necessary if the pressure assigned must to replace or to superimpose to calculated diagram.
- Color: Identifies the color of view of the diagram until the elevation z.

1.5.12 Modulus of subgrade reaction

It is possible (in the context of analysis with the finite element method) to manage the stiffness of the springs which schematize the soil. The environment for the management of the stiffness modules is the following:
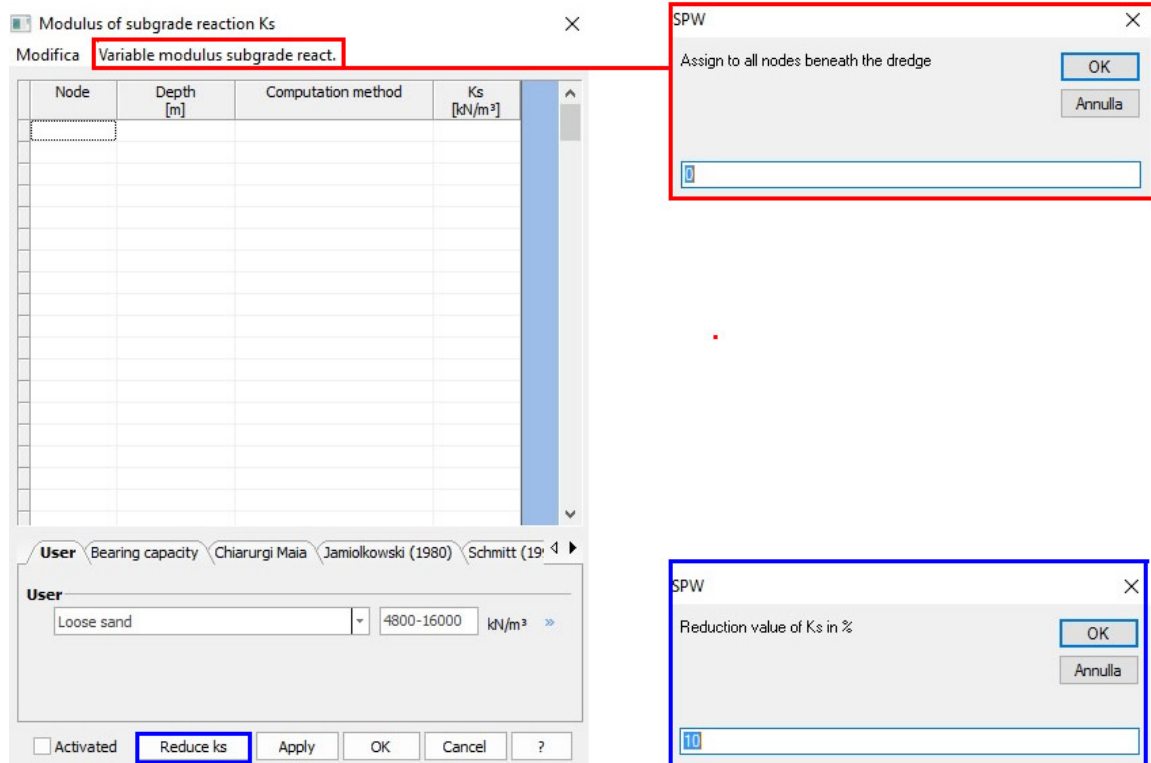
- Node: It is the node (within the finite element discretization) upon which to impose the assigned stiffness.
- Depth: It is the depth at which the node is positioned, or possibly the depth at which you want to fix the modulus of subgrade reaction.
- Computation method: It is the calculation method through which the modulus of subgrade reaction must be determined. The software allows choosing from the following options:

User

The user can select different soils, from the drop-down text box, each being associated to a range of values for the modulus of subgrade reaction. The user can enter a numerical value for the modulus of subgrade reaction and click the double blue arrow in order to assign the entered value.

Bearing capacity

According to the method which exploits the concepts of bearing capacity, the modulus of subgrade reaction is calculated on the basis of the following formula:

The user must enter the parameters As, Bs and n so as to allow the software to calculate the modulus of subgrade reaction applying the formula.

"The most general form of either a horizontal or a lateral modulus of subgrade reaction is:

$$K_s = A_s + B_s Z^n \qquad (9\text{-}10)$$

where: As = constant for either horizontal or vertical numbers

Bs = coefficient of depth variation

Z = depth of interest below ground

n = exponent to give ks the best fit (if load test or other data are available)

Either As or Bs in this equation may be zero; at ground surface As is zero for a lateral ks but at any small depth As > 0. For footings and mats (plates in general), As > 0 and Bs @ 0. Equation (9-10) can be used with the proper interpretation to the bearing-capacity equations of Table 4-1 (with di factor dropped) to give

$$q_{ult} = c \times N_c \times s_c + \gamma \times Z \times N_q \times s_q$$
$$+ 0.5 \times \gamma \times B \times N_\gamma \times s_\gamma \qquad (9\text{-}10a)$$

Observing that

$$A_s = C \times (c \times N_c \times S_c + 0.5 \times \gamma \times B \times N_\gamma \times s_\gamma)$$
$$\text{and} \qquad B_s \times Z^1 =$$
$$C \times (\gamma \times N_q \times s_q) Z^1$$

We obtain a ready means to estimate ks. In these equations the Terzaghi or Hansen bearing-capacity factors can be used. The C factor is 40 for SI units and 12 for Fps, using the same reasoning that qult occurs at a 0.0254-m and 1-in. settlement but with no SF, since this equation directly gives qult. Where there is concern that ks does not increase without boundwith depth Z, we may adjust the $B_s Z$ term by one of two simple methods:

Method 1: $B_s \times \tan^{-1}(Z/D)$

Method 2: $(B_s/D^n) \times Z^n = B'_s \times Z^n$

where

D = maximum depth of interest, say, the length of a pile

Z = current depth of interest

N = your best estimate of the exponent

Table 9-1 may be used to estimate a value of ks to determine the correct order of magnitude of the subgrade modulus obtained using one of the approximations given here. Obviously if a computed value is two or three time larger that the table ranger indicates, the computations should be rechecked for a possible gross error. Note, however, if you use a reduced value of displacement (say, 6 mm or 12 mm) instead of 0.0254 m you may well exceed the table range. Other than this, if no computational error (or a poor assumption) is found then use judgment in what value to use. The table values are intended as guides. The reader should not use, say, an average of the range given as a "good" estimate. "

Joseph E. Bowles (1997), "Foundation analysis and design"

TABLE 9-1
Range of modulus of subgrade reaction k_s

Use values as guide and for comparison when using approximate equations

Soil	k_s, kN/m³
Loose sand	4800–16 000
Medium dense sand	9600–80 000
Dense sand	64 000–128 000
Clayey medium dense sand	32 000–80 000
Silty medium dense sand	24 000–48 000
Clayey soil:	
$q_u \leq 200$ kPa	12 000–24 000
$200 < q_u \leq 800$ kPa	24 000–48 000
$q_u > 800$ kPa	> 48 000

Joseph E. Bowles (1997), "Foundation analysis and design"

TABLE 4-1
Bearing-capacity equations by the several authors indicated

Terzaghi (1943). See Table 4-2 for typical values and for $K_{p\gamma}$ values.

$$q_{ult} = cN_cs_c + \bar{q}N_q + 0.5\gamma BN_\gamma s_\gamma$$

$$N_q = \frac{a^2}{a\cos^2(45 + \phi/2)}$$

$$a = e^{(0.75\pi - \phi/2)\tan\phi}$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi} - 1\right)$$

For: strip   round   square
$s_c = 1.0$ 1.3   1.3
$s_\gamma = 1.0$ 0.6   0.8

Meyerhof (1963).* See Table 4-3 for shape, depth, and inclination factors.

Vertical load: $q_{ult} = cN_cs_cd_c + \bar{q}N_qs_qd_q + 0.5\gamma B'N_\gamma s_\gamma d_\gamma$

Inclined load: $q_{ult} = cN_cd_ci_c + \bar{q}N_qd_qi_q + 0.5\gamma B'N_\gamma d_\gamma i_\gamma$

$$N_q = e^{\pi\tan\phi}\tan^2\left(45 + \frac{\phi}{2}\right)$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = (N_q - 1)\tan(1.4\phi)$$

Hansen (1970).* See Table 4-5 for shape, depth, and other factors.

General:† $q_{ult} = cN_cs_cd_ci_cg_cb_c + \bar{q}N_qs_qd_qi_qg_qb_q + 0.5\gamma B'N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$

when $\phi = 0$

use $q_{ult} = 5.14s_u(1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$

N_q = same as Meyerhof above

N_c = same as Meyerhof above

$N_\gamma = 1.5(N_q - 1)\tan\phi$

Vesić (1973, 1975).* See Table 4-5 for shape, depth, and other factors.
Use Hansen's equations above.

N_q = same as Meyerhof above

N_c = same as Meyerhof above

$N_\gamma = 2(N_q + 1)\tan\phi$

*These methods require a trial process to obtain design base dimensions since width B and length L are needed to compute shape, depth, and influence factors.
†See Sec. 4-6 when $i_i < 1$.

Joseph E. Bowles (1997), "Foundation analysis and design"

Chiarurgi-Maia method

This method is used to calculate the modulus of subgrade reaction on the basis of the oedometric modulus, of the diameter of the piling and of Poisson's ratio. The formula applied to calculate the modulus of subgrade reaction using this method is the following:

$$k_s = \frac{E_{ed}}{d \cdot (1 - v^2)} \cdot \left(\frac{E_{ed} \cdot d^4}{EJ} \right)^{\frac{1}{12}}$$

In the previous formula, E_{ed} is the soil oedometric modulus, d is the diameter of the pile, n is Poisson's ratio ad EJ is the flexural stiffness of the piling.

Jamiolkowski's method

This method is used to calculate the modulus of subgrade reaction on the basis of the following parameters:

- secant modulus of elasticity, expressed in kPa
- dimensionless coefficient which depends on the bound model assumed for the calculation of the bulkhead (it is equal to 1 for free bulkheads)
- dimensionless coefficient which depends on the depth at which to calculate the modulus of subgrade reaction
- bulkhead embedment depth, expressed in m

In particular, Jamiolkowski's method refers to the secant modulus of elasticity of the ground corresponding to the activation of 50% of the limit pressure (Es,50). Therefore, for the calculation of the modulus of subgrade reaction, the following formula is applied:

$$k_s = \frac{E_{s,50}}{\rho \cdot t} \cdot C_p$$

In the previous formula, t is equal to the embedment depth and r is a dimensionless coefficient equal to 1 for free diaphragm on the foot or to the ratio of the position of the null displacement point beneath the dredge line to the embedment depth for the diaphragm with partial fixed joint on the foot. C_p is a dimensionless depth coefficient estimated using the following formula:

Schmitt's method

This method is used to calculate the modulus of subgrade reaction on the basis of the diameter of the section, of the modulus of elasticity of the ground and of the modulus of elasticity of the material constituting the

structure. In particular, Schmitt proposes to refer to the oedometric modulus of the ground E_{ed} as well as to the stiffness relevant to the supporting structure *(expressed through the characteristic length   of the Winkler beams)*, obtaining:

$$k = 2.1 \cdot \frac{E^{\frac{4}{3}}_{ed}}{EJ^{\frac{1}{3}}}$$

where E_{ed} is the edometric modulus of the ground, while EJ represents the flexural stiffness of the bulkhead.

Ménard's method

This method is used to calculate the modulus of subgrade reaction of the ground on the basis of the results of tests carried out using the Ménard pressuremeter. In particular, the modulus of subgrade reaction is estimated on the basis of the following factors:

- E_M, pressurmeter modulus, expressed in kN/m^2
- coefficient which takes into account the viscous behaviour (dimensionless coefficient)
- characteristic length, expressed in m

In particular, this method refers to the pressuremeter modulus of the ground E_M, obtained experimentally through a pressuremetric test:

$$k = \frac{E_M}{\alpha \cdot \dfrac{L}{2} + 0.13 \cdot (9 \cdot L)^{\alpha}}$$

Where a is   coefficient which takes into account the viscous behaviour of the ground, and L is a characteristic length that the author indicates as corresponding to 2/3 of the bulkhead embedment depth.

The Reduce ks button activates a dialog box where must be entered the percentage value of which is to reduce the subgrade reaction module.
It is possible to automatically assign a value to the modulus of subgrade reaction for all the nodes under the bottom of the excavation line by selecting Variable modulus of subgrade react. / Assign to all nodes beneath the dredge.
Click Apply for the software to take into consideration all data entered in this window.

1.5.13 Boundary conditions

In some situations there can be conditions – on the displacements, rotations or springs – which must be respected beforehand in the calculation procedure. In this case, we refer to an imposition of the boundary conditions. The software allows managing the boundary conditions in the following window:
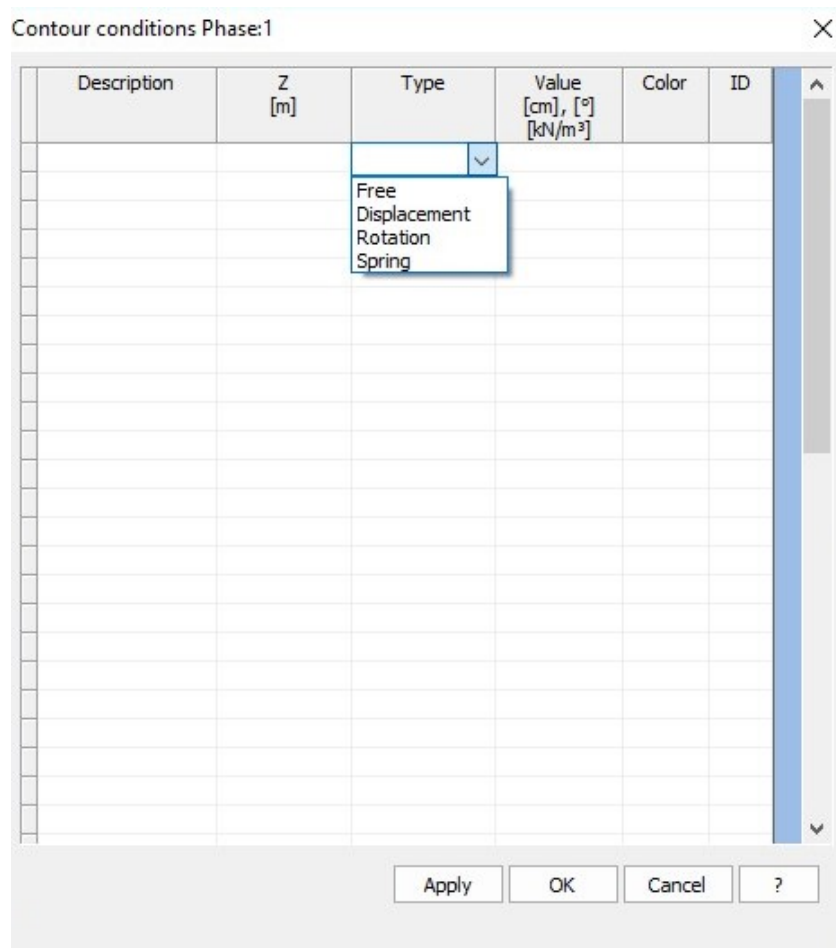
- Description: Identifies the boundary condition to be imposed by means of a name.
- Z: Expressed in m. It is the depth at which to impose the boundary condition.
- Type: It is the type of condition to be imposed. In SPW, the following boundary conditions can be managed:

  o Free: Through this condition, you can impose the node at z level to be free and consequently by no means bound.
  o Displacement: expressed in m. It is possible to impose – for a certain depth (z) – the displacement to be equal to the imposed one.

o Rotation: expressed in degrees. It is possible to impose – for a certain depth (z) – the rotation of the elastic line of the bulkhead to be equal to the imposed one.

o Spring: It is possible to insert a spring at the z depth which simulates an elastically yielding bound.

- Value: It is the value of the imposed boundary condition. The unit of measurement to be taken into consideration is the one associated to the type of imposed condition (cm for displacement, degrees for rotations, kN/m³ for the spring).
- Color: Color to be used for the display of the possible imposed condition.
- ID: It is the index which identifies univocally the imposed boundary condition.

1.5.14 Construction phases

SPW allows the insertion of multiple construction phases.
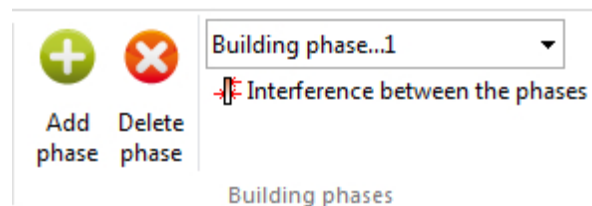
Add phase: Allows to enter an analysis phase at increasing depths.

Delete phase: Deletes the current analysis phase.

Interference between the phases: Allows to have interference between the phases. Initializes each phase according to the interaction with the previous phase. Even in the absence of anchors can be noticed a difference in stress between the phases, due to the interaction of the stress field of the current phase compared to the previous one.

The pressure diagrams will be related to the phase stress field. By activating this command, will be created automatically load conditions at the current phase based on the previous one.

If there is a difference between the phases of displacement, will activate the passive rods.

In the analysis by phases is not possible to automatically calculate the depth of embedment using finite elements.

N.B. If the command Interference between the phases is not active, in the calculation process there is no interference between the phases.

1.6 Computation

1.6.1 Analysis

The analysis of the bulkhead is organized by analysis phases and load combinations. In particular, it is possible to define more analysis phases, which differ from one another on the basis of the input data. For each analysis phase, it is possible to define more load combinations. The environment that manages the analysis of the bulkhead and related phases and load combinations is the following:

The main considerations to be made as regards the analysis procedure are the following:

- It is possible to analyze more constructive phases, and for each of them to analyze more load combinations. (To add a new load combination just click on New combination button, or click Remove to delete a combination previously selected. Click Regenerate combination to turn back to default combination.)
- To each load combination, it is possible to associate the type of checks to be carried out and consequently whether to carry out checks to ultimate

limit states (SLU) or serviceability limit states (SLE), and select the type GEO (geotechnical) STR (structural) and UPL, HYD (hydraulic).

- It is possible to manage the partial amplification factors for loads as well as the partial reduction factors for the the geotechnical parameters and for the resistances.
- The seismic coefficients kh and kv to be associated to the seismic combinations can be calculated automatically or inserted manually.

In order to carry out the analysis of the bulkhead (all phases and combinations) it is necessary to click on the Calculate button.

N.B. The combination coefficients and synthesis of the results reported in the Analysis dialog box refer to the combination that the user selects in the box shown in red in the previous image.

> ⚠ In order to consider in the analysis phase the loads inserted, the anchorage or support systems is necessary that the related combination factor is different from 0!

Summary of results

For each construction phase and for every combination is displayed a summary of the results, in particular, will be given the following values:

- o Embedment depth
- o Maximum earth pressure
- o Maximum moment
- o Maximum shear
- o Maximum horizontal displacement H based on which the user will determine if the value is acceptable to the bulkhead in question (the result is obtained in case of F.E.M. analysis)
- o Maximum vertical displacement V (the result is obtained in case of F.E.M. analysis)
- o Number of iterations performed (the result is obtained in case of F.E.M. analysis)
- o Vertical bearing capacity with safety factor FS (not determined for the generic sections of A, E, I, G input)

Results anchors and struts

In case of anchors and / or struts will also be shown the results for:

Anchors
- o Name given to the anchor in the input phase
- o Q: bearing capacity, in kN

o R: reaction of the anchor, in kN

o FS: safety factor calculated as:

$$FS = Q/R$$

Check satisfied for FS> 1.

N.B. The program calculates the resistance of the anchor considering the anchored part (bulb length), while the value of the reaction is determined according to the length of the free part.

Struts

o Name given to the strut in the input phase

o Reaction of the strut, in kN

o Safety factor calculated as:

$$FS = (Area * Fyd) / \text{Reaction of the strut}$$

The results are related to a linear meter of the bulkhead.

Additional results

By clicking the button highlighted in yellow in the image below, additional results are displayed:

The first table shows the values of the earth pressure, neutral pressure, the pressure due to load lines, the seismic pressure (see. Pressures diagrams), stress (see. Stress diagrams) and the subgrade reaction module for all the elements in which the bulkhead is discretized. If the modulus of subgrade reaction is not assigned by the user, will be determined automatically by the software.

The second table refers to the verification of the anchors and reports the average depths of the bulb, the thrust coefficient, the average pressure of the bulb, the bearing capacity of the soil, bond strength, the bearing capacity of the anchor and the failure mechanism.

1.6.2   Results of the structural analysis

The user can get immediate information on the structural analysis results of the bulkhead sections in terms of reinforcements, extreme deformations, result verification and so on.

The environment for the management of the results of the structural analysis is the following:

Z [m]	Section name	N [kN]	M [kNm]	T [kN]	No. of diameter bars	Nu [kN]	Mu [kNm]	Cond. of bending check	Bending check	Concrete max. strain	Steel max. strain	Neutral axis [cm]	Stirrups spacing [cm]	Shear strength [kN]	Shear safety OK<=1	Cond. of shear check	Ang. of incl. of the struts
0.4	CIR 0.4/l=0.8	9.08	-0.27	-4.83	8Ø16	9.07	-89.30	336.98	Verified	3.50E-03	-8.57E-03	-9.56	19.1Ø10	Concrete=235.27 Stirrups=222.73	1.00	Verified	21.80
0.8	CIR 0.4/l=0.8	11.91	-2.20	-13.98	8Ø16	11.90	-89.62	40.76	Verified	3.50E-03	-8.52E-03	-9.52	19.1Ø10	Concrete=236.85 Stirrups=222.73	1.00	Verified	21.80
1.2	CIR 0.4/l=0.8	13.94	-7.79	-27.97	8Ø16	13.94	-89.85	11.53	Verified	3.50E-03	-8.48E-03	-9.48	19.1Ø10	Concrete=237.12 Stirrups=222.73	1.00	Verified	21.80
1.6	CIR 0.4/l=0.8	15.97	-18.98	-45.61	8Ø16	15.97	-90.07	4.75	Verified	3.50E-03	-8.44E-03	-9.45	19.1Ø10	Concrete=237.38 Stirrups=222.73	1.00	Verified	21.80
2.0	CIR 0.4/l=0.8	18.01	-37.22	-43.63	8Ø16	18.00	-90.30	2.43	Verified	3.50E-03	-8.40E-03	-9.41	19.1Ø10	Concrete=237.65 Stirrups=222.73	1.00	Verified	21.80
2.4	CIR 0.4/l=0.8	20.04	-54.67	-14.87	8Ø16	20.04	-90.53	1.66	Verified	3.50E-03	-8.36E-03	-9.38	19.1Ø10	Concrete=237.92 Stirrups=222.73	1.00	Verified	21.80
2.8	CIR 0.4/l=0.8	21.27	-60.62	5.29	8Ø16	21.28	-90.67	1.50	Verified	3.50E-03	-8.34E-03	-9.36	19.1Ø10	Concrete=238.08 Stirrups=222.73	1.00	Verified	21.80
3.2	CIR 0.4/l=0.8	24.10	-58.50	10.67	8Ø16	24.11	-90.99	1.56	Verified	3.50E-03	-8.28E-03	-9.31	19.1Ø10	Concrete=238.46 Stirrups=222.73	1.00	Verified	21.80
3.6	CIR 0.4/l=0.8	25.34	-54.24	13.63	8Ø16	25.33	-91.13	1.68	Verified	3.50E-03	-8.26E-03	-9.28	19.1Ø10	Concrete=238.62 Stirrups=222.73	1.00	Verified	21.80
4.0	CIR 0.4/l=0.8	26.57	-48.79	15.34	8Ø16	26.56	-91.26	1.87	Verified	3.50E-03	-8.23E-03	-9.26	19.1Ø10	Concrete=238.78 Stirrups=222.73	1.00	Verified	21.80
4.4	CIR 0.4/l=0.8	27.80	-42.65	16.09	8Ø16	27.79	-91.40	2.14	Verified	3.50E-03	-8.21E-03	-9.24	19.1Ø10	Concrete=238.95 Stirrups=222.73	1.00	Verified	21.80
4.8	CIR 0.4/l=0.8	29.03	-36.21	16.10	8Ø16	29.04	-91.54	2.53	Verified	3.50E-03	-8.19E-03	-9.22	19.1Ø10	Concrete=239.11 Stirrups=222.73	1.00	Verified	21.80
5.2	CIR 0.4/l=0.8	30.27	-29.77	15.57	8Ø16	30.26	-91.67	3.08	Verified	3.50E-03	-8.17E-03	-9.20	19.1Ø10	Concrete=239.27 Stirrups=222.73	1.00	Verified	21.80
5.6	CIR 0.4/l=0.8	31.50	-23.55	14.65	8Ø16	31.50	-91.81	3.90	Verified	3.50E-03	-8.15E-03	-9.18	19.1Ø10	Concrete=239.44 Stirrups=222.73	1.00	Verified	21.80
6.0	CIR 0.4/l=0.8	32.73	-17.68	14.11	8Ø16	32.72	-91.94	5.20	Verified	3.50E-03	-8.13E-03	-9.16	19.1Ø10	Concrete=239.60 Stirrups=222.73	1.00	Verified	21.80
6.2	CIR 0.4/l=0.8	33.29	-15.12	13.28	8Ø16	33.29	-92.00	6.09	Verified	3.50E-03	-8.12E-03	-9.15	19.1Ø10	Concrete=239.67 Stirrups=222.73	1.00	Verified	21.80
6.4	CIR 0.4/l=0.8	33.85	-12.70	12.39	8Ø16	33.85	-92.07	7.25	Verified	3.50E-03	-8.11E-03	-9.14	19.1Ø10	Concrete=239.75 Stirrups=222.73	1.00	Verified	21.80
6.5	CIR 0.4/l=0.8	34.41	-10.45	11.48	8Ø16	34.41	-92.13	8.82	Verified	3.50E-03	-8.10E-03	-9.13	19.1Ø10	Concrete=239.82 Stirrups=222.73	1.00	Verified	21.80
6.7	CIR 0.4/l=0.8	34.97	-8.36	10.52	8Ø16	34.98	-92.19	11.02	Verified	3.50E-03	-8.09E-03	-9.12	19.1Ø10	Concrete=239.90 Stirrups=222.73	1.00	Verified	21.80
6.9	CIR 0.4/l=0.8	35.53	-6.45	9.56	8Ø16	35.54	-92.25	14.30	Verified	3.50E-03	-8.08E-03	-9.12	19.1Ø10	Concrete=239.97 Stirrups=222.73	1.00	Verified	21.80
7.1	CIR 0.4/l=0.8	36.09	-4.71	8.59	8Ø16	36.10	-92.31	19.59	Verified	3.50E-03	-8.07E-03	-9.11	19.1Ø10	Concrete=240.04 Stirrups=222.73	1.00	Verified	21.80
7.3	CIR 0.4/l=0.8	36.65	-3.15	7.51	8Ø16	36.66	-92.37	29.31	Verified	3.50E-03	-8.06E-03	-9.10	19.1Ø10	Concrete=240.12 Stirrups=222.73	1.00	Verified	21.80
7.5	CIR 0.4/l=0.8	37.21	-1.79	5.45	8Ø16	37.22	-92.43	51.74	Verified	3.50E-03	-8.05E-03	-9.09	19.1Ø10	Concrete=240.19 Stirrups=222.73	1.00	Verified	21.80
7.6	CIR 0.4/l=0.8	37.77	-0.80	3.28	8Ø16	37.78	-92.50	116.33	Verified	3.50E-03	-8.04E-03	-9.08	19.1Ø10	Concrete=240.27 Stirrups=222.73	1.00	Verified	21.80

The results that can be seen are the classic ones of a structural analysis (Stresses at ultimate limit state, maximum deformation, maximum stresses, neutral axis position, verification results etc.). The colors blue and red that highlight some of the results have no meaning for the verification itself, they only indicate which sections are most loaded with flexural reinforcement and shear.

N.B. These results are returned for the combinations that have been associated with type STR in the Analysis window.

1.6.3 Stress diagrams

The diagrams resulting from the stress analysis can be seen phase-by-phase and combination-by-combination. Are available diagrams for pressures, moment, shear and displacement (the last one only in the case of F.E.M. analysis).
The user can choose to view the diagrams for any analysis phase and for any load combination.
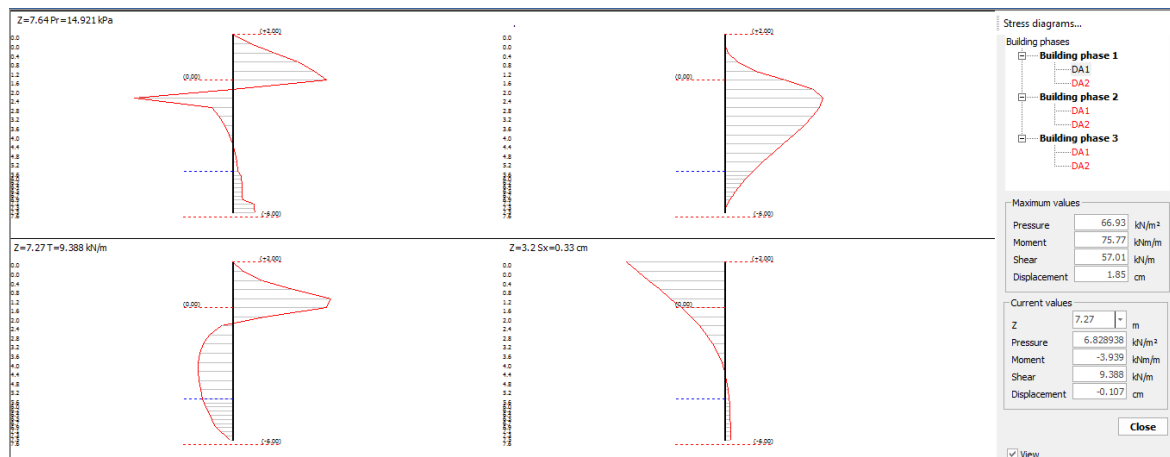
N.B. Right click on the diagrams to print or copy the image displayed.

1.6.4   Pressure diagram

Click on the Pressure diagram button in the Computation menu to see:
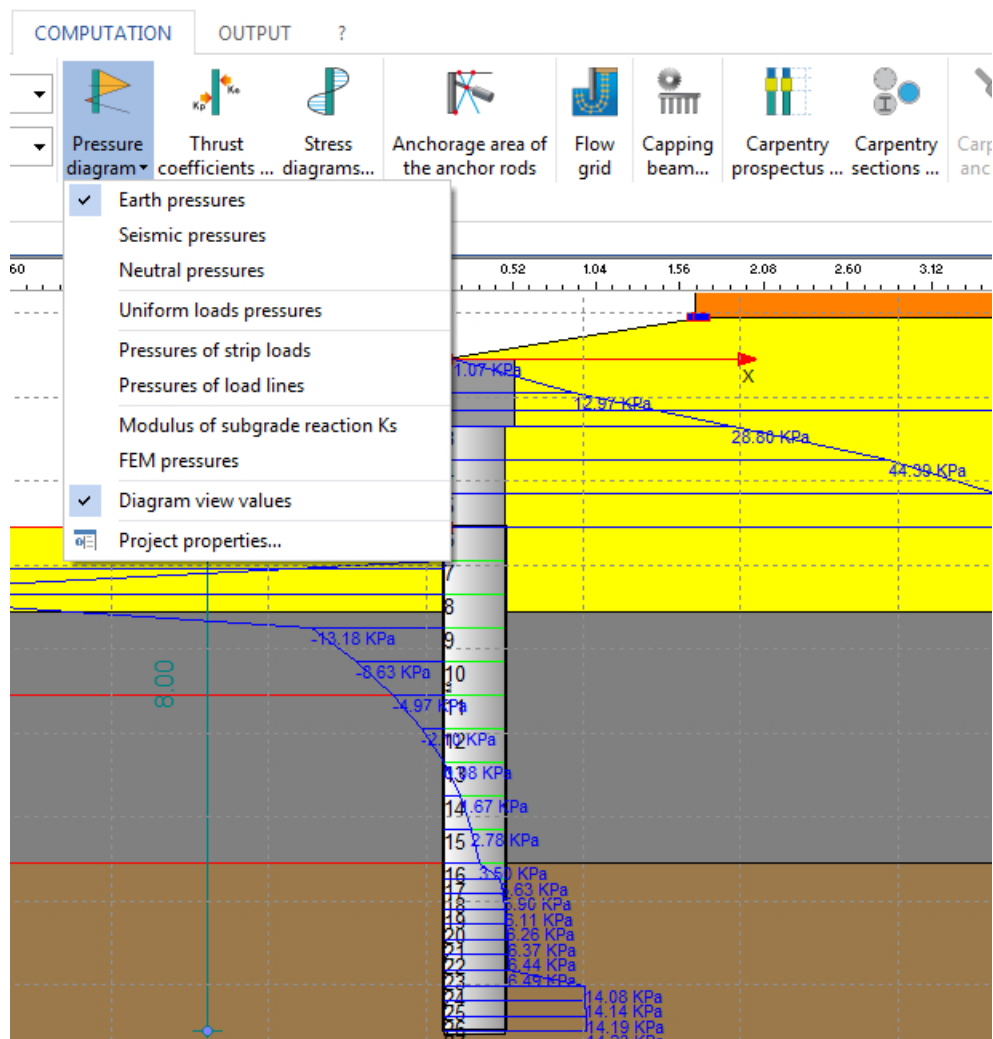
Earth pressures: Used to display the diagram of the earth pressures on the bulkhead.

Seismic pressures: Displays the increment of seismic pressures.

Neutral pressures: In the presence of groundwater, displays the neutral pressures diagram. It is only represented the diagram which acts on the excavation side and not on the embedded side.

Uniform loads pressures: Shows the diagram of the pressure generated by the presence of overloads.

Pressures of strip loads: Shows the pressure diagram generated by the presence of strip loads.

Pressures of load lines: Shows the pressure diagram generated by the presence of load lines.

FEM pressures: Displays the pressure diagram in the use of F.E.M.

Diagram view values: Shows, for all diagrams seen above, the value of pressure at different depths.

Project properties: Allows to define the general settings related to graphic design: height of the text, drawing fins size, level line offset.

N.B. To print the pressures diagrams use Print button in the File menu.

1.6.5 Fastening area of the anchors

The anchor, seen as an element aimed at integrating the strength resources of the bulkhead, is only useful if the anchoring occurs in stable areas of the ground. It is thus necessary to estimate with certainty the most suitable area in which to fasten the anchor. The principle according to which to calculate the anchoring area is to identify that ground surface where the active zone does not intersect with the passive zone. The procedure used in the software is the following:

- Identify – along the bulkhead, near (or beneath) the dredge line – the position of the point where the bending moment is nullified.
- Start from the point of null moment, trace out two lines inclined by 45°-j/2 and 45°+j/2, respectively (Rankine zones).
- At this point, position the anchor so that its end is in the dashed zone and beneath the AD line in the following figure, in order to obtain the utmost performance. If the anchoring end (fixed point) is positioned in the BCD zone, the anchoring wedge is limited to the BC line, but it cannot be obtained the maximum performance of the anchor.
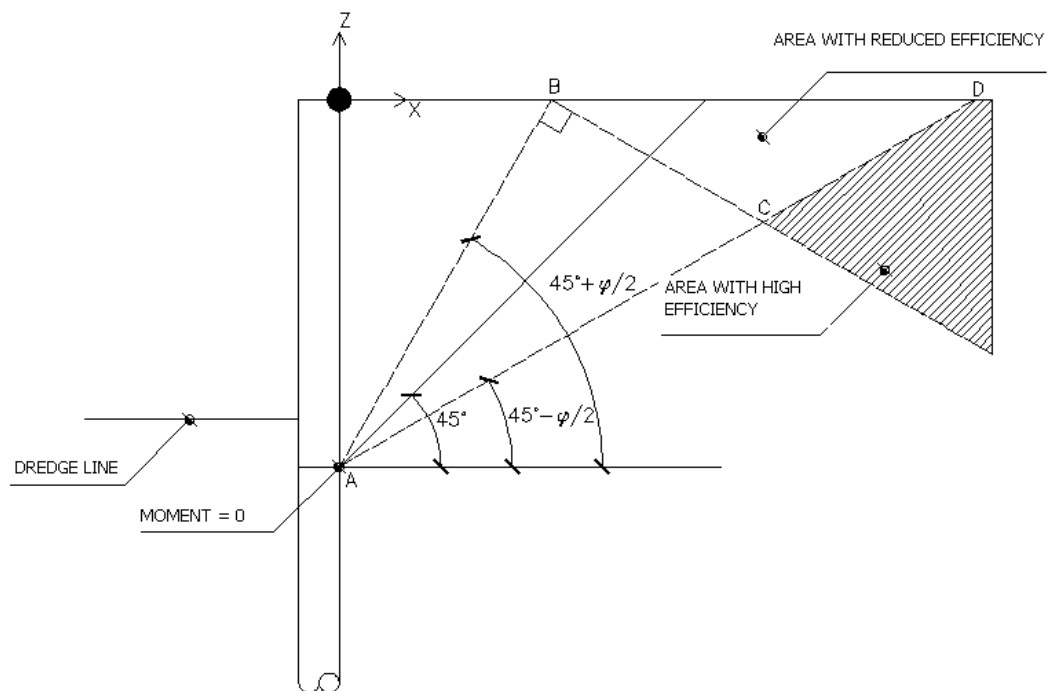
1.6.6   Continuous beam

The software provides a service program for the stress analysis of a continuous beam schematizing the capping beam or the anchoring beam. The environment for the use of the service program is the following:

The key data for the proper functioning of the program are:

- Geometry of the piling, in terms of the diameter of the piles, the distance between piles, etc.
- Mechanical properties (E and n) and geometrical properties (A and J) of the beam to be calculated.
- There can be inserted external bounds to the beam or external forces.
- The output is provided in terms of bending moment, normal stress and shear (for each abscissa).
- Are also calculated the maximum and minimum values of bending moment, Share and Displacement.

N.B. After assigning all required data, click the Generate model button, the model of the continuous beam will be displayed in the work area and after clicking the Calculate button the program provides the stresses.

1.6.7 Global stability

Starts the procedure for calculation of the global stability of the model. The ground-work global stability is performed automatically by the program for each phase and combination. For each stability calculation it is possible to select a number of options such as: type of calculation limit states or limit equilibrium, author, shape of the surface, etc.

.

Global stability	⟊ ×
◢ **General data**	
Building phase	1 Building phase
Combination	A2+M2+R1 ▾
Embedment depth	6
Horizontal seismic coefficient	0
Vertical seismic coefficient	0
Safety factor	1.1
Computation author	Bishop
Surface shape	Circular
Analysis type	Limit states
Number of strips	10
◢ **Ground parameters**	
Angle of shearing resistance	1.25
Effective cohesion	1.25
Undrained strength	1.4
Unit weight	1
◢ **Slope file**	
File	C:\GEOSTRU 2018\SLOPE\Slop

Combination

| Execute | Regenerate | Close | ? |

For the proper operation of the global stability module you must have installed the software Slope and in the File section set the path that defines the location of the executable file of the software Slope. Clicking the Execute button it will start the software Slope/SPW.

From the menu Computation it can be selected the method to use for the analysis of global stability and start the calculation through the Perform Analysis command.
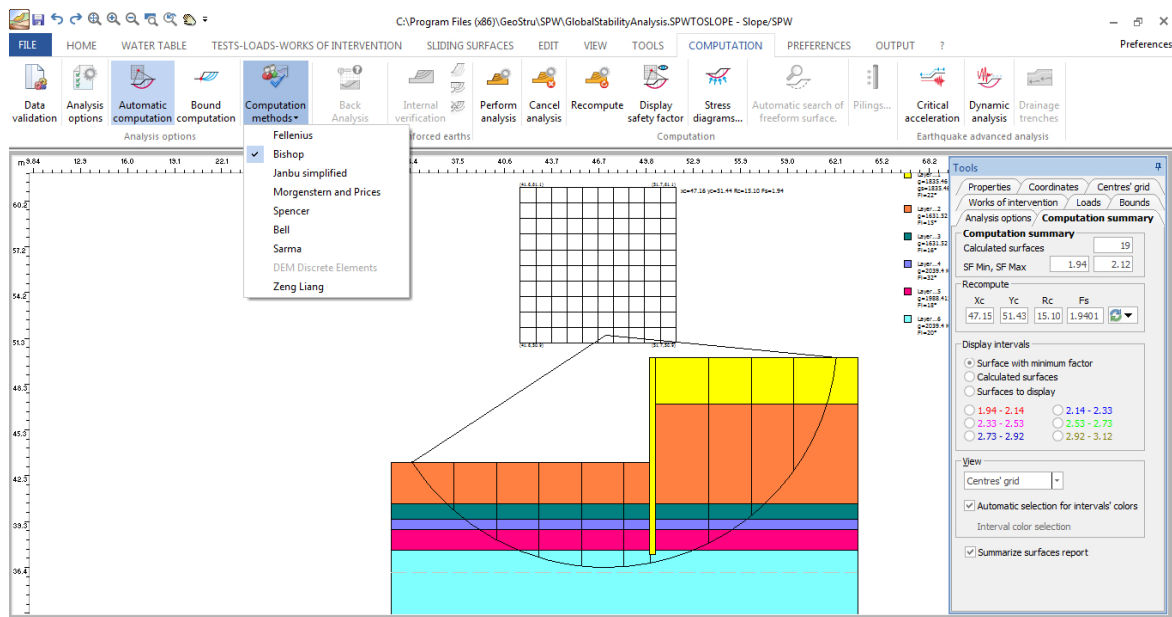
In the Computation summary tab (on the right side of the work area) is shown the minimum value of the safety factor to be compared with the degree of security considered acceptable.

The report for the global stability can be made in Slope > Output menu > Create report option.

N.B. For further information regarding the global stability, please see the manual of Slope software.

1.7   Output

1.7.1   Export DXF

The software allows exporting results in dxf format (i.e. the generation of a graphic printout). For exporting in dxf format, just click on the Export Dxf button from the Output menu. A window will open, allowing you to select the path where to save the dxf file. In the export file you can manipulate the reinforcement and the geometrical model of the calculation.
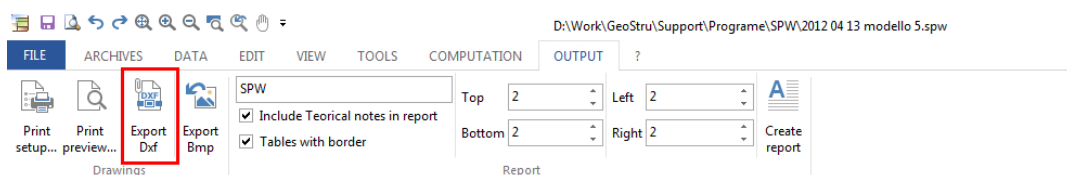
1.7.2 Computation report

The software allows exporting results in *.doc format (i.e. the generation of the calculation report). For the export in *.doc format just to click on the Create report button in the Output menu.
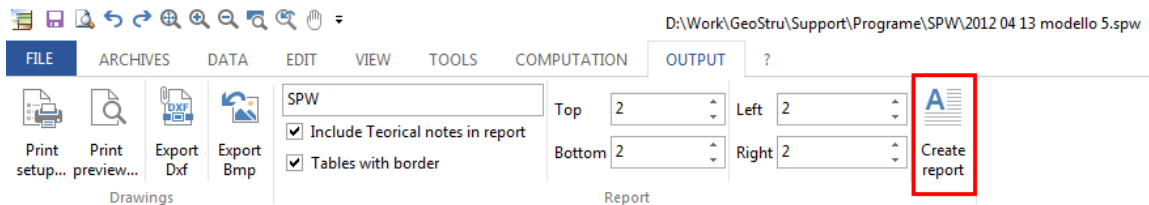
A window will open, allowing you to select the parts of the report to be printed:
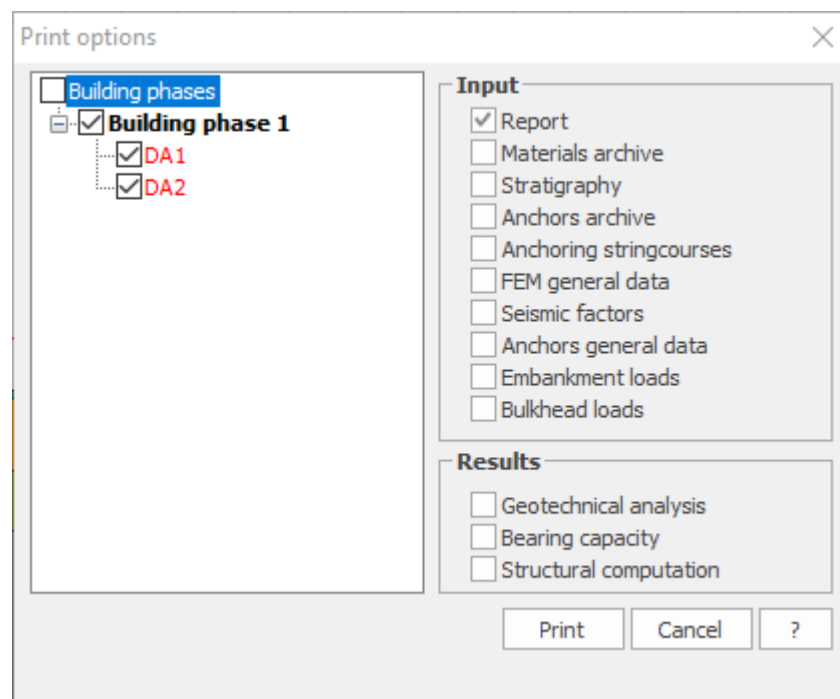
As you can easily see, the selection of the parts to be printed can be made with reference to the constructive phases, to the combinations, to the data and to the results. As a result, a streamlined document, smooth-flowing but at the same time significant with regard to the subject of the report, will be printed.
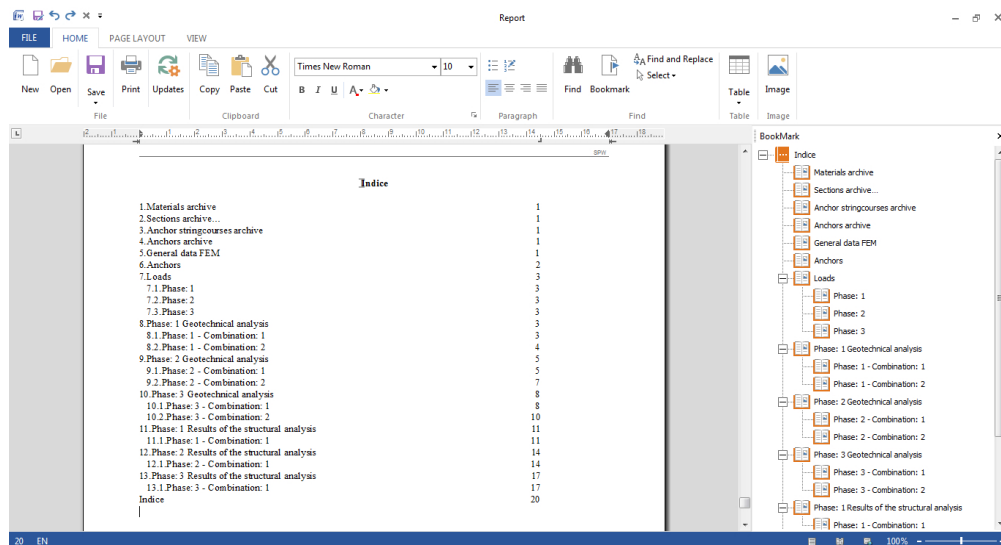
1.8 Preferences

Options

Display the dialog for setting the parameters relating to the worksheet, output, company data and autosaving.

Select language

Allows you to select the language from those available.

1.9 Theoretical notes

In the Computation report section are presented all the theoretical explanations: the computation method of the pressures, finite element and limit equilibrium solutions, analysis in the presence of groundwater, the calculation of structural elements etc.

1.9.1 LEM Method

The limit equilibrium method consists in looking for solutions that are compatible only with the static aspect of the problem. Basically, we reason in terms of equilibrium of a rigid body, without minding the kinematic congruence of the displacements. The main calculation diagrams we will refer to are the following:

- Overhanging bulkhead
- Anchored bulkhead with free end
- Anchored bulkhead with fixed end

Overhanging bulkhead

Calculation of the limit embedment depth

For unanchored bulkhead, the stability is guaranteed by the passive strength of the ground situated downhill from it; from the equilibrium of the moments as to the center of rotation, we obtain:

$$S_m \cdot B_m - R_v \cdot B_v = 0$$

Where the symbols have the following meaning:

S_m = horizontal component of the active thrust

B_m = arm of S_m as to O center of rotation

R_v = horizontal component of the passive strength

B_v = arm of R_v as to O center of rotation

Each term is a function of t, where t is the depth of the rotation center with respect to the downhill reference level (downhill natural surface level). The necessary length to guarantee the equilibrium of the horizontal translation is obtained by increasing t as follows:

$$t' = a \cdot t$$

$$d = t + t' = t \cdot (1 + a)$$

where a=0.2 (Blum method)
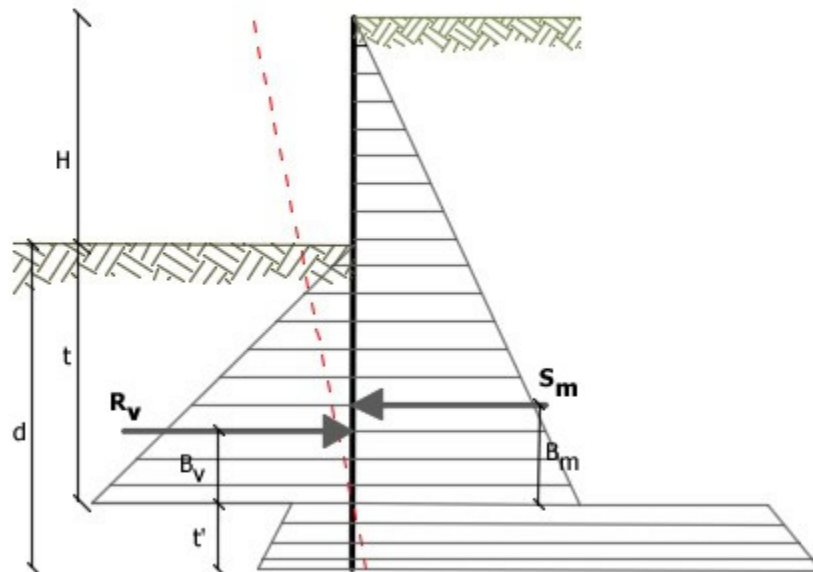
Reference diagram for calculating the equilibrium of the bulkhead

Safety factor on the passive strength

The length of the embedment d, as determined above, refers to the limit condition of an incipient collapse through a coefficient F. It is possible to introduce a safety margin on the passive strengths; such reduction is made as follows:

$$S_m \cdot B_m - \frac{R_v}{F} \cdot B_v = 0$$

Anchored bulkhead with free end

Calculation of the limit embedment depth

The stability of the work is also guaranteed by the anchors fastened upon the bulkhead. In order to use the calculation scheme with free end, the bulkhead must be sufficiently short and rigid. The embedment length will be determined by imposing the equilibrium to the rotation on the origin of the anchor indicated as B1

$$S_m \cdot (H + t - B_m - t_m) - R_v \cdot (H + t - B_v - t_m) = 0$$

Where the symbols have following meaning:

S_m = horizontal component of the active thrust

H = height of the ground to be supported

t = calculated embedment depth

B_m = arm of S_m as to the base of the bulkhead

P_m = ordinate of the point of application of the upstream anchor

R_v = horizontal component of the passive strength

B_v = arm of R_v

When t is known, S_m and R_v as well as the stress of the anchor will be determined.

Safety factor F on the passive strengths
The embedment length will be further increased so as to have a safety margin in working conditions by means of the safety factor F:

$$S_m \cdot (H + t - B_m - t_m) - \frac{R_v}{F} \cdot (H + t - B_v - t_m) = 0$$

Anchored bulkhead with fixed end

Calculation of the limit embedment depth

If the deepest section of the bulkhead does not translate or rotate, it can be assimilated to a fixed joint; in this case, the bulkhead is defined with fixed end. A procedure developed by BLUM allows obtaining the

embedment depth (t+t'), imposing the kinematic conditions of null displacements on the base of the work and on the origin of the anchor (B1) and the static conditions of null moment and shear on the base of the bulkhead. We therefore reach a 5th degree equation in (t+t') which can be easily solved.

Safety factor F on the strengths

In order to increase the safety factor, values of the reduced passive strengths have been introduced in the numerical developments.

1.9.2   FEM Method

The finite-elements method is the method which is based more than any other one upon solid and rational theoretical bases. The method presupposes that the problem is faced taking into account both the static aspect (and consequently the equilibrium of the problem) and the kinematic aspect (and consequently the congruence of the displacements or rather of the deformations). In the FEM method, the bulkhead is modeled like a set of beams (beam elements), with continuity bound to the ground by means of elastic springs, whose stiffness is estimated according to the elastic properties of the ground. In the following figure is schematically shown the model used for the analysis with finite-elements:
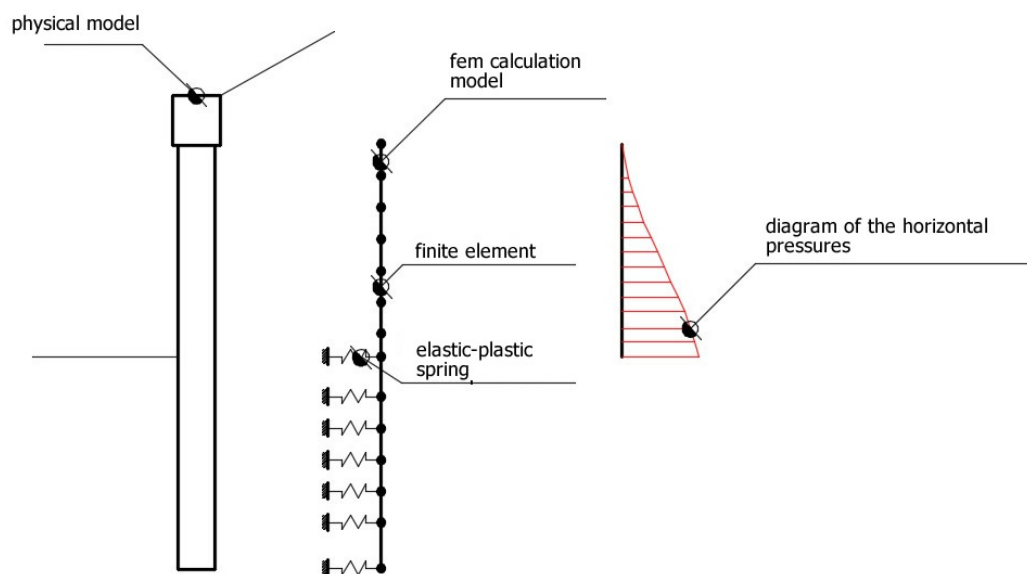
Schematization of the bulkhead-ground complex by means of finite-elements

The finite-elements method requires the knowledge of the ground and structure properties (unlike the LEM method, in which a condition of equilibrium for the rigid body is imposed). In particular, it is necessary to

know the modulus of subgrade reaction of the ground. The main parts composing the application of the method are the following:

- Characterization of the structure materials by means of:
    - evaluation of the active and passive thrust coefficients as well as of the modulus of subgrade reaction for the ground
    - evaluation of the flexural stiffness – shearing and normal – of the bulkhead
- Discretization of the bulkhead in finite elements and modeling of the springs:
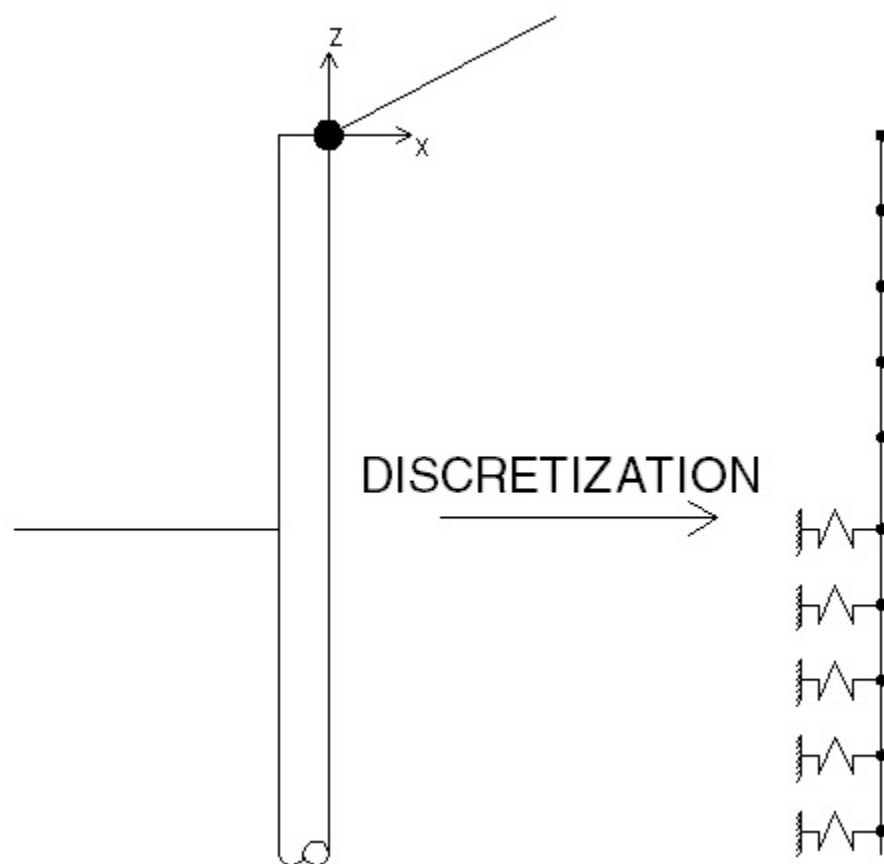
Discretization procedure

- Assembly of the contributions of each finite element

In this phase, the problem is reported in global, assembling the stiffness contributions of each finite element.

- Solution of the problem

The problem with finite elements is posed in terms of non-linear system in displacement variables. In this phase, an iterative procedure is set so as to allow the solution to a non-linear problem of the following kind:

In the previous equation, K(u) is the stiffness matrix of the problem (non-linear type), u is the displacement field that is the solution of the problem, and p is the vector of the external loads (associated to the distribution of ground pressures).

- Assembly and interpretation of the solution

Once the values of the u vector components have been determined, all information (relevant to the structural analysis) can be derived. In particular, the following parameters will be determined:

> o Moment
> o Shear
> o Normal stress

1.9.3   Anchors limit load

The anchors contribute to the increase in strength resources for the bulkhead. The calculation of the limit load of an anchor must be made considering thee different collapse mechanisms. In fact, the collapse may happen by pull-out off of the bulb, by pull-out off of the steel part of the concrete which contains it or, possibly, by breaking of the anchor (reaching the threshold of the steel strength). The procedure used in the software calculates the limit load with regard to all three collapse mechanisms, and defines as limit load of the anchor the minimum between the three collapse mechanisms.

Limit load for collapse in adherence to the bulb-ground interface

In this case, the limit load is calculated using the following formula (Schneebeli):

In the previous formula, the meaning of the symbols is the following:

- σ'_n is the effective stress acting in the center of the anchoring bulb
- K is a coefficient which represents – on the average and for the whole length – the interaction between the bulb and the ground (see table)
- A_b is the surface of the bulb in contact with the soil, evaluated using the following formula:

$$A_b = \pi \cdot D \cdot L_b$$

where D is the diameter of the bulb and Lb is the length of the bulb

- ca is the adhesion to the interface between the bulb and the ground. Please look at the figure contained in the Anchors section for a better comprehension of the symbols.

Table: values provided in the work of Prof. Carlo Cestelli Guidi
"Geotecnica e tecnica delle fondazioni" ("Geotechnics and foundation engineering"), Vol. 2,
Ed. Hoepli, year 1980.

Limit load for collapse in adherence to the steel-bulb interface

In this case, the shear stress of ultimate adherence between steel and concrete is involved. The formula which expresses the collapse load of the system is the following:

$$N_u = \pi \cdot D_a \cdot L_b \cdot \tau_{ud}$$

In the formula, the symbols have the following meaning:

- D_a is the diameter of the reinforcement which constitutes the anchor
- L_b is the length of the fastening bulb
- τ_{ud} is the adherence shear stress between steel and concrete

The design adherence shear stress f_{bd} is:

$$f_{bd} = f_{bk} / \gamma_C$$

where:

- $_c$ is the partial safety coefficient related to concrete, equal to 1.5
- f_{bk} is characteristic shear resistance given by:

$$f_{bk} = 2{,}25 \times \eta \times f_{ctk}$$

where:

- $\eta = 1{,}0$ for bars of diameter $\phi < 32$ mm
- $\eta = (132 - \phi)/100$ for bars with bigger diameter

In case of very thickened reinforcements or anchorages in zone of stressed concrete, the adherence strength is reduced by diving it at least by 1.5.

Limit load for collapse by reinforcement failure

This is a purely structural check and concerns the circumstance in which the stress in the steel reaches the failure limit stress. In this case, the collapse load is calculated using the following formula:

$$N_u = \pi \cdot \frac{D^2{}_a}{4} \cdot F_{yd}$$

In the previous formula, the symbols have the following meaning:

- D_a is the diameter of the reinforcement which constitutes the anchor
- Fyd is the calculation yield stress of steel

Once the collapse loads have been calculated for the three different mechanisms, the limit load of the anchor can be determined using the following formula:

$$N^T{}_u = \min(\min(N_u^{at}, N_u^{ac}), N^r{}_u)$$

Where the symbols have the following meaning:

- N_u^T is the ultimate limit load of the anchor
- N_u^{at} is the ultimate limit load which occurs for collapse in adherence between the bulb and the ground
- N_u^{ac} is the ultimate limit load which occurs for collapse in adherence between the bulb and the steel
- N_u^r is the ultimate limit load which occurs for collapse of the anchor's steel

For each anchorage, the failure mechanism is returned: ground failure, adherence resistance, steel resistance

1.9.4   Vertical settlements

For the calculation of the vertical settlements upstream of the bulkhead is used the *'Constant Volume'* method.

It is assumed that the variation of the total volume of the soil involved in the deformation process is null (deforming, the soils tend to dilate). In this case, once defined the thrust wedge, there is equality between the volume that occupies the ground after the movement of the bulkhead and the one the land frees for vertical settlements upstream work.

Assumes as thrust angle to the horizontal active 45+ϕ/2. Respect to the vertical 45-ϕ/2.
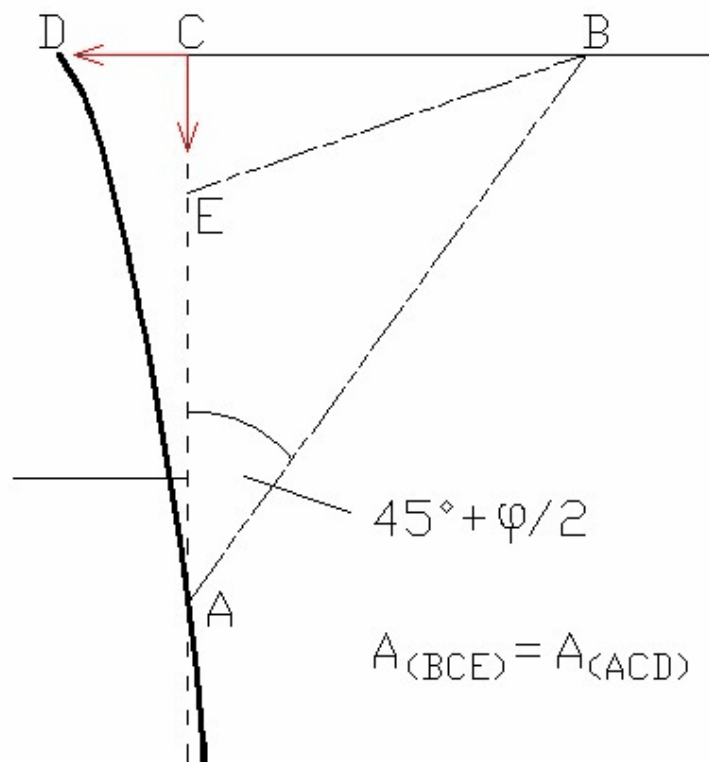
Scheme of reference for the calculation of the vertical settlements

1.9.5   Filtration

The term filtration indicates that physical phenomenon for which occurs the passage of water from an area with a given energy to other area with a lower energy, through a porous medium. The energy can be expressed as the sum of kinetic energy related to the velocity of the fluid, of the potential energy depending on the position of the point and of the pressure

of the liquid at the same point. Since the rate of filtration is always very small the kinetic term is negligible. In studying the filtration of water can appear problems, both of permanent and unsteady motion flow. With reference to the pressure of water, which plays an important role in most of the stability problems, please note that in permanent motion it remains constant over time, while in unsteady flow is a function of time and it may rise or fall.

With reference to the amount of water in the phenomenon of filtration through a certain area, please note that in continuous operation the amount of' water that enters is equal to that coming out, while under unsteady flow there is no equality and the difference represents the volume of water that is accumulated or expelled from the ground in the time interval considered. In the phenomenon of consolidation, which is a particular condition of unsteady flow, also interferes the compressibility of the soil. In steady state the area in which filtration develops, in the scheme of representation that is adopted, has two types of boundaries: one is the place where is known the water load and defines the border or boundary condition of the potential; the other one is an outline of waterproof materials, such as impermeable rock, clay, etc. which delimits the layer in which the filtration occurs and is defined as border or boundary condition of the water flow. To clarify what was said just remember for example the conditions of water flow in the constant load permeability test. In this test clearly the boundaries of the potential are the surfaces of the entry and exit of water from the soil sample. Because the container walls are impermeable, the flow is parallel to the container and the wall forms the boundary of the water flow:
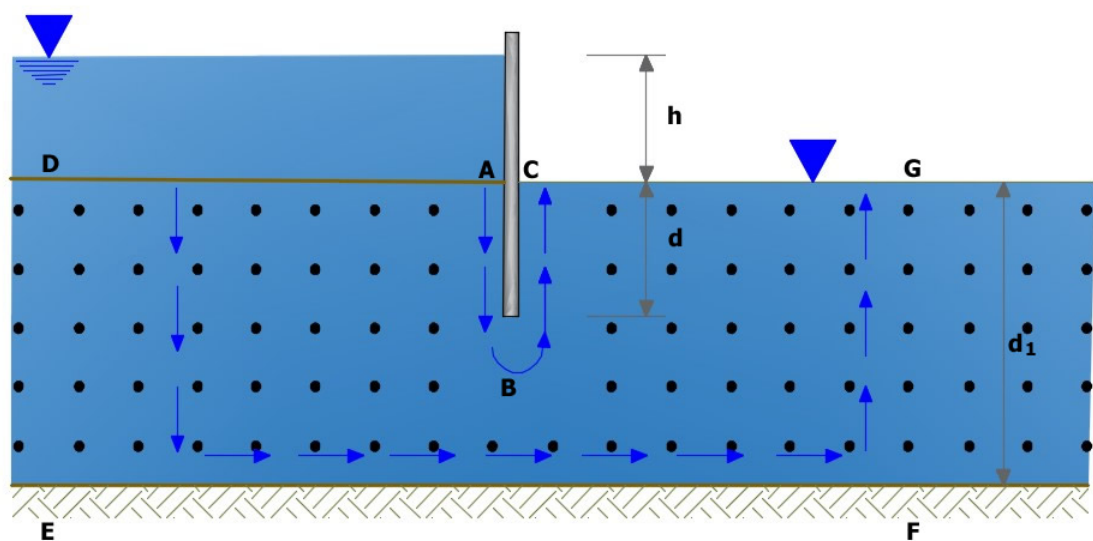
Scheme for the water flow

A practical case is the sheet piling (previous figure) that maintains a constant level h of water and which is driven into the soil to a depth d in a homogeneous layer of permeable soil (sand or gravel) of thickness dI, which rests on a impermeable layer (rock or clay). In this case we are talking about a confined flow, since the boundary conditions of the region in which the motion takes place are geometrically defined. The water flow is caused by the hydraulic load h; on the surface AD acts a constant load and this surface is the first boundary of the potential in our problem; on CG the load is also constant and this constitutes the second boundary. Obviously, to fulfill its task, the sheet piling must be impermeable, so its surface ABC is one of the boundaries of the flow while the surface EF of the impermeable layer will form the other border. Obviously in principle if the characteristics of water, soil and the impermeable layer, upstream and downstream of the sheet piling, are constant it can be considered that the points D, E, F and G are endlessly; in practice it is generally considered that the length concerned is included within 4-5 times the thickness of the layer. To determine the amount of water that seeps into the ground will make the assumption that the water flow is governed by Darcy's law and that the soil is homogeneous, isotropic and incompressible:

$$v = -k \cdot i$$

Darcy's law is valid for laminar flow, a condition that occurs for certain values of the Reynolds number, R. The value of R, that characterizes the flow passing from laminar to turbulent, takes different values depending on the authors; Taylor (1948) has indicated as a criterion for the validity of Darcy's law R <= 1. Other researchers have examined, especially for clay, the connection between the flow conditions and the hydraulic gradient; in particular Tavenas ed al. (1983) have come to the conclusion that, with regard to the clays, the Darcy's law is valid for gradients between 0.1 and 50.

To calculate the flow rate of filtration through the soil it is useful to determine the distribution the pore water pressure via the construction of the flow grid, that is, the system of streamlines and equipotential lines representing the flow of water through an incompressible soil. Accepting the hypothesis of incompressible ground for filtration motion in plane and steady conditions the continuity equation can be written in the form:

The two components of the velocity of the liquid, according to Darcy's law, can be expressed in the form:

$$v_x = -k \cdot i_x = -k \cdot \frac{\partial h}{\partial x}$$

$$v_z = -k \cdot i_z = -\frac{\partial h}{\partial z}$$

Combining these three equations is obtained:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial z^2}$$

which is the Laplace equation for permanent motion on a plane, on the assumption of homogeneous, isotropic and incompressible material. This equation can be expressed by means of two conjugate functions   and   . Indeed, we can express the velocity components as partial derivatives with respect to x and z of the function   = k h :

$$v_x = -\frac{\partial \varphi}{\partial x}$$

$$v_z = -\frac{\partial \varphi}{\partial z}$$

Then we can also write:

$$\frac{\partial^2 \varphi}{\partial x^2} + \frac{\partial^2 \varphi}{\partial z^2}$$

The existence of the function   = k h, according to velocity potential for a fluid in motion, implies null vorticity and that the motion is irrotational. We can then say that it is a function of stream such that:

$$v_x = \frac{\partial \psi}{\partial z}$$

So we have:

$$\frac{\partial \varphi}{\partial x} = \frac{\partial \psi}{\partial z}$$

$$\frac{\partial \varphi}{\partial z} = -\frac{\partial \psi}{\partial x}$$

And we can also write:

and   are known, respectively, as a function of potential and function of stream. Taking up the case before indicated, the water that seeps through the soil at the bottom of a pile wall (figure below) we notice that two equipotential lines are the surfaces of the upstream and downstream ground of the same sheet piling; Furthermore the surface of the impermeable layer is a stream line or a flow line. By solving the Laplace equation in accordance with these boundary conditions, we can construct the flow net. Each strip between two adjacent flow lines is a flow channel and each part of the flow channel comprised between two equipotential lines is a field. It is therefore convenient to build the equipotential lines in a manner that the piezometric height difference between two successive lines is constant and the flow lines in such a way that each flow channel has a constant flow. If h is the total hydraulic load and Na is the total number of identified piezometric height differences, the difference in hydraulic load between two successive equipotential lines is:

$$\Delta h = \frac{h}{N_a}$$

In a point z as shown in the following figure the pressure is:
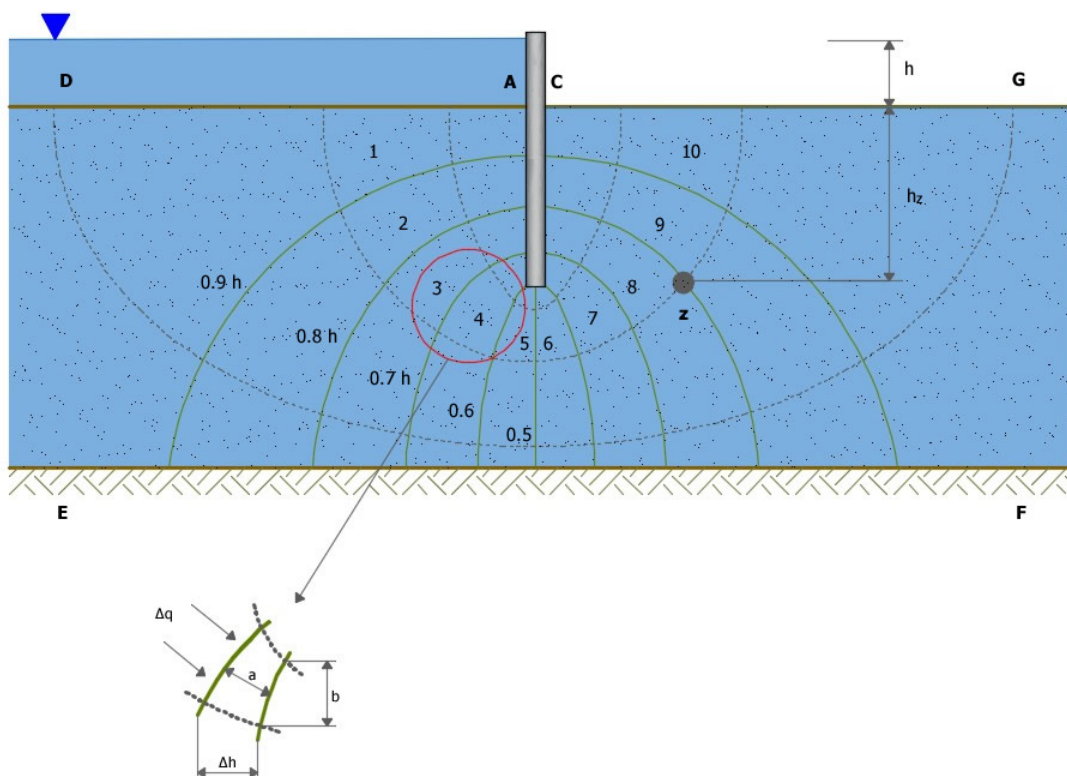
Schematization of the flow net

Being n the number of piezometric height differences crossed to arrive in z, in the example on the previous figure we have:

$$\frac{n}{N_a} = \frac{8}{10}$$

If there were no flow of water, that is, if the downstream surface were impermeable, the hydrostatic pressure at this point would be:

$$p = (h + h_z) \cdot \gamma_w$$

since the water moves, there is a loss of load which according to the filtration net drawn at the point z is equal to 8/10h. The excess water pressure at the point z is then given by:

$$\left(h - \frac{8}{10} \cdot h \right) \cdot \gamma_w = \frac{2}{10} \cdot h \cdot \gamma_w$$

To know the extent of filtration flow we consider a field, that is, an area between two flow lines and two equipotential lines; the length of the side in the direction of the flow lines is a and therefore the hydraulic gradient in a field is:

$$i = \frac{\Delta h}{a}$$

and the velocity:

$$v = k \cdot i = k \cdot \frac{\Delta h}{a} = \frac{k}{a} \cdot \frac{h}{N_a}$$

Assuming that the other side of the field is of length b, then the flow through the field per length unit of sheet piling will be:

$$\Delta q = v \cdot b = \frac{k}{a} \cdot \frac{h}{N_a} \cdot b$$

for each flow tube; if b = a, that is, if the elements of the filtration net are square, we get:

If N_b is the total number of flow channels of the total flow per length unit of sheet piling will be:

$$Q = \sum \Delta q = k \cdot h \cdot \frac{N_b}{N_a}$$

In this way, when it is constructed the flow net, one can easily calculate the flow rate. The filtration net is often built with experimental methods in the laboratory, with analog models graphically or by trial. In complex situations of the subsoil, for succession of layers and anisotropy of permeability, one can get the filtration net by means of numerical methods (FEM, BEM, finite difference method).

1.9.6   Siphoning check

In the presence of water and in conditions of initiation of a filtration motion, in the ground is generated a force of direct filtration upwards which can cancel the weight of the soil and, if this is lacking cohesion, can drag the particles and produce a collapse of the ground. This collapse mechanism is known as siphoning; it manifests itself in the outlet point of the first flow line, namely that in adherence to the bulkhead. Another phenomenon to check in flow conditions is the lift of the dredge.

Vertical effective stress in the presence of hydraulic gradient

In the presence of a hydraulic gradient i, the effective vertical stress is calculated with the following formula:

$$\sigma'_v = z \cdot \left(\gamma' - i \cdot \gamma_w \right)$$

in the above formula the meaning of the symbols is the following:

- γ' is the unit weight of effective ground
- i is the hydraulic gradient
- γ_w is the unit weight of water

The vertical stress in annulled when:

The safety factor for siphoning is the ratio between the critical gradient i_c and the efflux gradient i_E

$$F_S = \frac{i_c}{i_E}$$

1.9.7 Uplift check

In the case of a diaphragm driven into the ground, the presence of the water in positions such as to trigger a filtration motion involves the establishment of a filtration force which, if directed upwards, may cancel the weight of the soil which, in the absence of cohesion, it can be dragged by the water flow and affect the stability of the work. The phenomenon of dredge stability, similar to that of siphoning, has been faced for the first time by Terzaghi (1943). Unlike the siphoning, which is a localized phenomenon in the outlet point of the first flow line, the dredge uplift extends to a depth equal to bulkhead embedment depth for a width equal to half of that embedment.
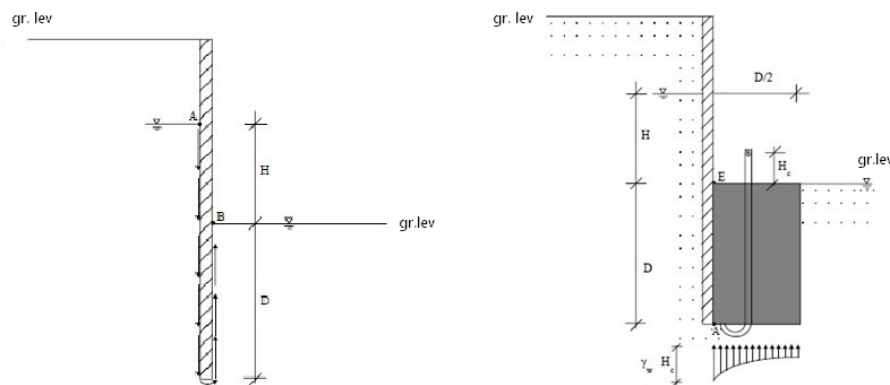
To simplify the problem of determining the actual results from porewater pressure at point A, it is assumed that the value of the overpressure to the foot of the diaphragm is constant over the length D/2 and equal to $_w$ x H_c. H_c is used to determine the expression of the efflux gradient i_E:

From which is obtained:

The filtration force S_w which tends to lift the block of soil involved is equal to:

$$S_w = H_c \times \gamma_w \times \frac{D}{2}$$

The limit conditions of stability are reached when S_w equals the effective weight of the block, therefore, the dredge uplift safety factor is defined as the ratio between the effective weight of the block and the filtration force:

$$F_s = \frac{W'}{S_w} = \frac{\gamma' \times {D^2}/{2}}{H_c \times \gamma_w \times \frac{D}{2}} = \frac{\gamma' \times D}{H_c \times \gamma_w}$$

2 Standards

2.1 Eurocodes

2.1.1 Eurocode 7

EN 1997 Eurocode 7 introduces in the verifications regarding structural and geotechnical limit states design approaches that vary for different combinations of groups partial coefficients for actions, for material strength and overall strength of the system.

Each EU member state issues the National Annex (NA) or detailed specifications for the application of the directives contained in EN 1997.

For example, the first approach is used in the UK and Portugal, the second approach in most European countries (Germany, Slovakia, Italy, etc.) for the calculation of the bearing capacity and the third approach in the Netherlands and in most European countries for the calculation of slope stability.

The specifications give the values of the partial factors to be used and indicate approaches to be adopted in the design phase for the different works (bearing capacity, anchors, bulkheads, retaining walls, etc.).

DESIGN APPROACHES

2.4.7.3.4.2 Design Approach 1

1. Except for the design of axially loaded piles and anchors, it shall be verified that a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:

Combination 1: A1 "**+**" M1 "**+**" R1

Combination 2: A2 "**+**" M2 "**+**" R1

where "+" implies: "to be combined with".

NOTE In Combinations 1 and 2, partial factors are applied to actions and to ground strength parameters.

2. For the design of axially loaded piles and anchors, it shall be verified that a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors:

Combination 1: A1 "+" M1 "+" R1
Combination 2: A2 "+" (M1 or M2) "+" R4

NOTE 1 In Combination 1, partial factors are applied to actions and to ground strength parameters. In Combination 2, partial factors are applied to actions, to ground resistances and sometimes to ground strength parameters.
NOTE 2 In Combination 2, set M1 is used for calculating resistances of piles or anchors and set M2 for calculating unfavourable actions on piles owing e.g. to negative skin friction or transverse loading.

3. If it is obvious that one of the two combinations governs the design, calculations for the other combination need not be carried out. However, different combinations may be critical to different aspects of the same design.

2.4.7.3.4.3 Design Approach 2
1. It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination of sets of partial factors:

Combination: A1 "+" M1 "+" R2

NOTE 1 In this approach, partial factors are applied to actions or to the effects of actions and to ground resistances.
NOTE 2 If this approach is used for slope and overall stability analyses the resulting effect of the actions on the failure surface is multiplied by γ_E and the shear resistance along the failure surface is divided by $\gamma_{R;e.}$

2.4.7.3.4.4 Design Approach 3
1. It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination of sets of partial factors:

Combination: (A1* or A2†) "+" M2 "+" R3

*on structural actions
†on geotechnical actions

NOTE 1 In this approach, partial factors are applied to actions or the effects of actions from the structure and to ground strength parameters.
NOTE 2 For slope and overall stability analyses, actions on the soil (e.g. structural actions, traffic load) are treated as geotechnical actions by using the set of load factors A2.

The table 3.1. below shows which of partial factor are used in each design approach, depending on the type of structure being designed.

Structure	Partial factors sets used in Design Approach...			
	1		2	3
	Combination 1	Combination 2		
General	A1+M1+R1	A2+**M2**+R1	**A1+R2**+M1	**A1***($\underline{A2}^+$)+**M2**+R3
Slope	A1+M1+R1	A2+**M2**+R1	**E1+R2**+M1	E2+**M2**+R3
Piles and anchor-ages	**A1**+M1+$\underline{R1}$	A2+M1+**R4**	**A1**+$\underline{R2}$+M1	**A1***($\underline{A2}^+$)+**M2**+$\underline{R3}$

Table 3.1 - *Ultimate limit state, design approach (*on structural actions,+ on geotechnical actions)*

Design Approach 1			Combination 1			Combination 2		
			A1	M1	R1	A2	M2	R1
Permanent actions (G)	Unfavorable	γ_G	1,35			1,0		
	Favorable	$\gamma_{G,fav}$	1,0			1,0		
Variable actions (Q)	Unfavorable	γ_Q	1,5			1,3		
	Favorable	$\gamma_{Q,fav}$	0			0		
Coef.of shearing resistance (tanϕ)		γ_f		1,0			1,25	
Effective cohesion (c')		$\gamma_{c'}$		1,0			1,25	
Undrained strength (cu)		γ_{cu}		1,0			1,4	
Unconfined compressive strength (q_u)		γ_{qu}		1,0			1,4	
Weight density ( )		γ_g		1,0			1,0	

Resistance (R)	γ_R			1,0		1,0

Table 3.2 - Shows the relative magnitude of the key parameters when using Combination and using Combination 2

Design Approach 2			A1	M1	R1
Permanent actions (G)	Unfavorable	γ_G	1,35		
	Favorable	$\gamma_{G,fav}$	1,0		
Variable actions (Q)	Unfavorable	γ_Q	1,5		
	Favorable	$\gamma_{Q,fav}$	0		
Material properties(c)		γ_M		1,0	
Material resistance (Rv)		γ_{Rv}			1,4
Sliding resistance (Rh)		γ_{Rh}			1,1
Earth resistance against retaining structures		γ_{Re}			1,4
....in slope					1,1

Table 3.3 - Shows the relative magnitude of the key parameters when using Design Approach 2

Design Approach 3			A1	A2	M2	R3
Permanent actions (G)	Unfavorable	γ_G	1,35	1,0		
	Favorable	$\gamma_{G,fav}$	1,0	1,0		
Variable actions (Q)	Unfavorable	γ_Q	1,5	1,3		
	Favorable	$\gamma_{Q,fav}$	0	0		
Coeff.of shearing resistance (tanϕ)		γ_f			1,25	
Effective cohesion (c')		$\gamma_{c'}$			1,25	
Undrained strength (cu)		γ_{cu}			1,4	
Unconfined compressive strength (q_u)		γ_{qu}			1,4	
Weight density ( )		γ_g			1,0	
Resistance (R) (except for pile shaft in tension)		γ_R				1,0
Pile shaft resistance in tension		$\gamma_{R,st}$				1,1

Table 3.4 - Shows the relative magnitude of the key parameters when using Design Approach 3

Spread foundations

6.1 General
1. The provisions of this Section apply to spread foundations including pads, strips and rafts.

2. Some of the provisions may be applied to deep foundations such as caissons.

6.2 Limit states

1. The following limit states shall be considered and an appropriate list shall be compiled:

- loss of overall stability;
- bearing resistance failure, punching failure, squeezing;
- failure by sliding;
- combined failure in the ground and in the structure;
- structural failure due to foundation movement;
- excessive settlements;
- excessive heave due to swelling, frost and other causes;
- unacceptable vibrations.

6.3 Actions and design situations

1. Design situations shall be selected in accordance with 2.2.
2. The actions listed in 2.4.2(4) should be considered when selecting the limit states for calculation.
3. If structural stiffness is significant, an analysis of the interaction between the structure and the ground should be performed in order to determine the distribution of actions.

6.4 Design and construction considerations

1. When choosing the depth of a spread foundation the following shall be considered:

- reaching an adequate bearing stratum;
- the depth above which shrinkage and swelling of clay soils, due to seasonal weather
  changes, or to trees and shrubs, may cause appreciable movements;
- the depth above which frost damage may occur;
- the level of the water table in the ground and the problems, which may occur if excavation
  for the foundation is required below this level;
- possible ground movements and reductions in the strength of the bearing stratum by
  seepage or climatic effects or by construction procedures;
- the effects of excavations on nearby foundations and structures;
- anticipated excavations for services close to the foundation;
- high or low temperatures transmitted from the building;
- the possibility of scour;
- the effects of variation of water content due to long periods of drought, and subsequent

periods of rain, on the properties of volume-unstable soils in arid climatic areas;

- the presence of soluble materials, e.g. limestone, claystone, gypsum, salt rocks;

2. Frost damage will not occur if:
> - the soil is not frost-susceptible;
> - the foundation level is beneath frost-free depth;
> - frost is eliminated by insulation.

3. EN-ISO 13793:2001 may be applied for frost protecting measures for building foundations.

4. In addition to fulfilling the performance requirements, the design foundation width shall take account of practical considerations such as economic excavation, setting out tolerances, working space requirements and the dimensions of the wall or column supported by the foundation.

5. One of the following design methods shall be used for spread foundations:
> - a direct method, in which separate analyses are carried out for each limit state. When checking against an ultimate limit state, the calculation shall model as closely as possible the failure mechanism, which is envisaged. When checking against a serviceability limit
>   state, a settlement calculation shall be used;
> - an indirect method using comparable experience and the results of field or laboratory measurements or observations, and chosen in relation to serviceability limit state loads so as to satisfy the requirements of all relevant limit states;
> - a prescriptive method in which a presumed bearing resistance is used (see 2.5).

6. Calculation models for ultimate and serviceability limit state design of spread foundations on soil given in 6.5 and 6.6 respectively should be applied. Presumed bearing pressures for the design of spread foundations on rock should be applied according to 6.7.

6.5 Ultimate limit state design

6.5.1 Overall stability

1. Overall stability, with or without the foundations, shall be checked particularly in the following situations:
> - near or on a natural or man-made slope;
> - near an excavation or a retaining wall;

- near a river, a canal, a lake, a reservoir or the sea shore;

- near mine workings or buried structures.

2. For such situations, it shall be demonstrated using the principles described in Section 11, that a stability failure of the ground mass containing the foundation is sufficiently improbable.

6.5.2 Bearing resistance

6.5.2.1 General

1. The following inequality shall be satisfied for all ultimate limit states:

$$V_d \leq R_d \qquad [6.1]$$

2. R_d shall be calculated according to 2.4.

3. Vd shall include the weight of the foundation, the weight of any backfill material and all earth pressures, either favorable or unfavorable. Water pressures not caused by the foundation load shall be included as actions.

6.5.2.2 Analytical method

1. The sample analytical calculation for bearing resistance given in Annex D may be used.

2. An analytical evaluation of the short-term and long-term values of Rd shall be considered, particularly in fine-grained soils.

3. Where the soil or rock mass beneath a foundation presents a definite structural pattern of layering or other discontinuities, the assumed rupture mechanism and the selected shear strength and deformation parameters shall take into account the structural characteristics of the ground.

4. When calculating the design bearing resistance of a foundation on layered deposits, the properties of which vary greatly between one another, the design values of the ground parameters shall be determined for each layer.

5. Where a strong formation underlies a weak formation, the bearing resistance may be calculated using the shear strength parameters of the weak formation. For the reverse situation, punching failure should be checked.

6. Analytical methods are often not applicable to the design situations described in 6.5.2.2(3)P, 6.5.2.2(4)P and 6.5.2.2(5). Numerical procedures should then be applied to determine the most unfavorable failure mechanism.

7. The overall stability calculations described in Section 11 may be applied.

6.5.2.3 Semi-empirical method

1. The sample semi-empirical method for bearing resistance estimation using pressuremeter test results given in Annex E is recommended.

6.5.2.4 Prescriptive method using presumed bearing resistance

1. The sample method for deriving the presumed bearing resistance for spread foundations on rock given in Annex G is recommended. When this method is applied, the design result should be evaluated on the basis of comparable experience.

6.5.3 Sliding resistance

1. Where the loading is not normal to the foundation base, foundations shall be checked against failure by sliding on the base.
2. The following inequality shall be satisfied:

$$H_d \leq S_d + E_{pd} \qquad [6.2]$$

3. H_d shall include the design values of any active earth forces imposed on the foundation.
4. R_d shall be calculated according to 2.4.
5. The values of R_d and $R_{p;d}$ should be related to the scale of movement anticipated under the limit state of loading considered. For large movements, the possible relevance of post-peak behaviour should be considered. The value of $R_{p;d}$ selected should reflect the anticipated life of the structure.
6. For foundations bearing within the zone of seasonal movements of clay soils, the possibility that the clay could shrink away from the vertical faces of foundations shall be considered.
7. The possibility that the soil in front of the foundation may be removed by erosion or human activity shall be considered.
8. For drained conditions, the design shear resistance, R_d , shall be calculated either by factoring the ground properties or the ground resistance as follows;

$$R_d = V'_d \tan \phi_d \quad (6.3a)$$

or

$$R_d = (V'_d \tan \phi_k) / \gamma_{R;h} \quad (6.3b)$$

Note In design procedures where the effects of actions are factored, the partial factor for the actions (γ_F) is 1,0 and $V'_d = V'_k$ in equation (6.3b).

9. In determining V_d', account shall be taken of whether H_d and V'_d are dependent or independent actions.
10. The design friction angle ϕ_d may be assumed equal to the design value of the effective critical state angle of shearing resistance, $\phi'_{cv;d}$, for cast-in-situ concrete

foundations and equal to 2/3 $?'_{cv;d}$ for smooth precast foundations. Any effective cohesion c' should be neglected.

11. For undrained conditions, the design shearing resistance, R_d , shall be calculated either by factoring the ground properties or the ground resistance as follows:

$$R_d = A_c \, c_{u;d} \text{ (6.4a)}$$

or

$$R_d = (A_c \, c_{u;k}) / \, _{R;h} \text{ (6.4b)}$$

12. If it is possible for water or air to reach the interface between a foundation and an undrained clay subgrade, the following check shall be made:

$$R_d \leq 0,4 \, V_d \text{ (6.5)}$$

13. Requirement (6.5) may only be disregarded if the formation of a gap between the foundation and the ground will be prevented by suction in areas where there is no positive bearing pressure.

6.5.4 Loads with large eccentricities

1. Special precautions shall be taken where the eccentricity of loading exceeds 1/3 of the width of a rectangular footing or 0,6 of the radius of a circular footing. Such precautions include:
   - careful review of the design values of actions in accordance with 2.4.2;
   - designing the location of the foundation edge by taking into account the magnitude of construction tolerances.

2. Unless special care is taken during the works, tolerances up to 0,10 m should be considered.

6.5.5 Structural failure due to foundation movement

1. Differential vertical and horizontal foundation displacements shall be considered to ensure that they do not lead to an ultimate limit state occurring in the supported structure.

2. A presumed bearing pressure may be adopted (see 2.5) provided displacements will not cause an ultimate limit state in the structure.

3. In ground that may swell, the potential differential heave shall be assessed and the foundations and structure designed to resist or accommodate it.

6.6 Serviceability limit state design
6.6.1 General

1. Account shall be taken of displacements caused by actions on the foundation, such as those listed in 2.4.2(4).
2. In assessing the magnitude of foundation displacements, account shall be taken of comparable experience, as defined in 1.5.2.2. If necessary, calculations of displacements shall also be carried out.
3. For soft clays, settlement calculations shall always be carried out.
4. For spread foundations on stiff and firm clays in Geotechnical Categories 2 and 3, calculations of vertical displacement (settlement) should usually be undertaken. Methods that may be used to calculate settlements caused by loads on the foundation are given in 6.6.2.
5. The serviceability limit state design loads shall be used when calculating foundation displacements for comparison with serviceability criteria.
6. Calculations of settlements should not be regarded as accurate. They merely provide an approximate indication.
7. Foundation displacements shall be considered both in terms of displacement of the entire foundation and differential displacements of parts of the foundation.
8. The effect of neighboring foundations and fills shall be taken into account when calculating the stress increase in the ground and its influence on ground compressibility.
9. The possible range of relative rotations of the foundation shall be assessed and compared with the relevant limiting values for movements discussed in 2.4.9.

6.6.2 Settlement

1. Calculations of settlements shall include both immediate and delayed settlement.
2. The following three components of settlement should be considered for partially or fully saturated soils:

    - s_0 : immediate settlement; for fully-saturated soil due to shear deformation at constant volume, and for partially-saturated soil due to both shear deformation and volume reduction;

    - s_1 : settlement caused by consolidation;

    s_2 : settlement caused by creep.

3. The sample methods for evaluating settlements s_0 and s_1 given in Annex F may be applied.
4. Special consideration should be given to soils such as organic soils and soft clays, in which settlement may be prolonged almost indefinitely due to creep.
5. The depth of the compressible soil layer to be considered when calculating settlement should depend on the size and shape of the foundation, the variation in soil stiffness with depth and the spacing of foundation elements.

6. This depth may normally be taken as the depth at which the effective vertical stress due to the foundation load is 20 % of the effective overburden stress.

7. For many cases this depth may also be roughly estimated as 1 to 2 times the foundation width, but may be reduced for lightly-loaded, wider foundation rafts.

Note This approach is not valid for very soft soils.

8. Any possible additional settlement caused by self-weight compaction of the soil shall be assessed.

9. The following should be considered:
   - the possible effects of self-weight, flooding and vibration on fill and collapsible soils;
   - the effects of stress changes on crushable sands.

10. Either linear or non-linear models of the ground stiffness shall be adopted, as appropriate.

11. To ensure the avoidance of a serviceability limit state, assessment of differential settlements and relative rotations shall take account of both the distribution of loads and the possible variability of the ground.

12. Differential settlement calculations that ignore the stiffness of the structure tend to be over-predictions. An analysis of ground-structure interaction may be used to justify reduced values of differential settlements.

13. Allowance should be made for differential settlement caused by variability of the ground unless it is prevented by the stiffness of the structure.

14. For spread foundations on natural ground, it should be taken into account that some differential settlement normally occurs even if the calculation predicts uniform settlement only.

15. The tilting of an eccentrically loaded foundation should be estimated by assuming a linear bearing pressure distribution and then calculating the settlement at the corner points of the foundation, using the vertical stress distribution in the ground beneath each corner point and the settlement calculation methods described above.

16. For conventional structures founded on clays, the ratio of the bearing capacity of the ground, at its initial undrained shear strength, to the applied serviceability loading should be calculated (see 2.4.8(4)). If this ratio is less than 3, calculations of settlements should always be undertaken. If the ratio is less than 2, the calculations should take account of non-linear stiffness effects in the ground.

2.1.2 Eurocode 8

3 GROUND CONDITIONS AND SEISMIC ACTION (EC8 - part 1)

3.1 Ground conditions

3.1.2 Identification of ground types

1. Ground types A, B, C, D, and E, described by the stratigraphic profiles and parameters given in Table 3.1 and described hereafter, may be used to account for the influence of local ground conditions on the seismic action. This may also be done by additionally taking into account the influence of deep geology on the seismic action.

Note: The ground classification scheme accounting for deep geology for use in a country may be specified in its National Annex, including the values of the parameters S, TB, TC and TD defining the horizontal and vertical elastic response spectra in accordance with 3.2.2.2 and 3.2.2.3.

Ground type	Description of stratigraphic profile	Vs_{30} (m/s)	N_{SPT} (blows/30 cm)	cu (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800		
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with vs values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with vs > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft	<100 (indicative)		10-20

Ground type	Description of stratigraphic profile	Vs$_{30}$ (m/s)	N$_{SPT}$ (blows/30 cm)	cu (kPa)
	clays/silts with a high plasticity index (PI > 40) and high water content			
S$_2$	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or $\mathcal{S}$1			

Prospect 3.1-Ground types

2. The site should be classified according to the value of the average shear wave velocity, $v_{s,30}$, if this is available. Otherwise the value of NSPT should be used.

3. The average shear wave velocity $v_{s,30}$ should be computed in accordance with the following expression:

$$v_{s,30} = \frac{30}{\displaystyle\sum_{i=1,N} \frac{h_i}{v_i}}$$ (3.1)

where hi and vi denote the thickness (in meters) and shear-wave velocity (at a shear strain level of 10–5 or less) of the i-th formation or layer, in a total of N, existing in the top 30 m.

4. For sites with ground conditions matching either one of the two special ground types S1 or S2 , special studies for the definition of the seismic action are required. For these types, and particularly for S2 , the possibility of soil failure under the seismic action shall be taken into account.

Note: Special attention should be paid if the deposit is of ground type $\mathcal{S}$1 . Such soils typically have very low values of vs , low internal damping and an abnormally extended range of linear behaviour and can therefore produce anomalous seismic site amplification and soil-structure interaction effects (see EN 1998-5:2004, Section 6). In this case, a special study to define the seismic action should be carried out, in order to establish the dependence of the response
spectrum on the thickness and vs value of the soft clay/silt layer and on the stiffness contrast between this layer and the underlying materials.

3.2 Seismic action
3.2.1 Seismic zones

1. For the purpose of EN 1998, national territories shall be subdivided by the National Authorities into seismic zones, depending on the local hazard. By definition, the hazard within each zone is assumed to be constant.

2. For most of the applications of EN 1998, the hazard is described in terms of a single parameter, i.e. the value of the reference peak ground acceleration on type A ground, a_{gR}. Additional parameters required for specific types of structures are given in the relevant Parts of EN 1998.

Note: The reference peak ground acceleration on type A ground, a_{gR}, for use in a country or parts of the country, may be derived from zonation maps found in its National Annex.

3. The reference peak ground acceleration, chosen by the National Authorities for each seismic zone, corresponds to the reference return period TNCR of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years, PNCR) chosen by the National Authorities (see 2.1(1)P). An importance factor I equal to 1,0 is assigned to this reference return period. For return periods other than the reference (see importance classes in 2.1(3)P and (4)), the design ground acceleration on type A ground ag is equal toagR times the importance factor I (ag =gI x agR). (See Note to 2.1(4)).

4. In cases of low seismicity, reduced or simplified seismic design procedures for certain types or categories of structures may be used.

Note: The selection of the categories of structures, ground types and seismic zones in a country for which the provisions of low seismicity apply may be found in its National Annex. It is recommended to consider as low seismicity cases either those in which the design ground acceleration on type A ground, ag , is not greater than 0,08g (0,78 m/s²), or those where the product a_g x S is not greater than 0,1 g (0,98 m/s²). The selection of whether the value of a_g, or that of the product a_g x S will be used in a country to define the threshold for low seismicity cases, may be found in its National Annex.

5. In cases of very low seismicity, the provisions of EN 1998 need not be observed.

Note: The selection of the categories of structures, ground types and seismic zones in a country for which the EN 1998 provisions need not be observed (cases of very low seismicity) may be found in its National Annex. It is recommended to consider as very low seismicity cases either

those in which the design ground acceleration on type A ground, a_g, is not greater than 0,04g (0,39 m/s²), or those where the product a_g x S is not greater than 0,05g (0,49 m/s²). The selection of whether the value of a_g, or that of the product a_g x S will be used in a country to define the threshold for very low seismicity cases, can be found in its National Annex.

3.2.2 Basic representation of the seismic action
3.2.2.1 General
1. Within the scope of EN 1998 the earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an "elastic response spectrum".
2. The shape of the elastic response spectrum is taken as being the same for the two levels of seismic action introduced in 2.1(1)P and 2.2.1(1)P for the no-collapse requirement (ultimate limit state – design seismic action) and for the damage limitation requirement.
3. The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum.
4. For the three components of the seismic action, one or more alternative shapes of response spectra may be adopted, depending on the seismic sources and the earthquake magnitudes generated from them.

3.2.2.2 Horizontal elastic response spectrum
1. For the horizontal components of the seismic action, the elastic response spectrum Se(T) is defined by the following expressions (see Figure. 3.1):

$$0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 2,5 - 1) \right] \qquad (3.2)$$

$$T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \qquad (3.3)$$

$$T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \cdot \frac{T}{T_B} \qquad (3.4)$$

$$T_D \leq T \leq 4(s): S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \cdot \left[\frac{T_C \cdot T_D}{T} \right] \qquad (3.5)$$

where:

Se(T)   is the elastic response spectrum;

T   is the vibration period of a linear single-degree-of-freedom system;

ag   is the design ground acceleration on type A ground (ag = IagR);

TB   is the lower limit of the period of the constant spectral acceleration branch;

TC   is the upper limit of the period of the constant spectral acceleration branch;

TD   is the value defining the beginning of the constant displacement response range of the spectrum;

S   is the soil factor;

    is the damping correction factor with a reference value of   = 1for 5% viscous damping, see (3) of this subclause.
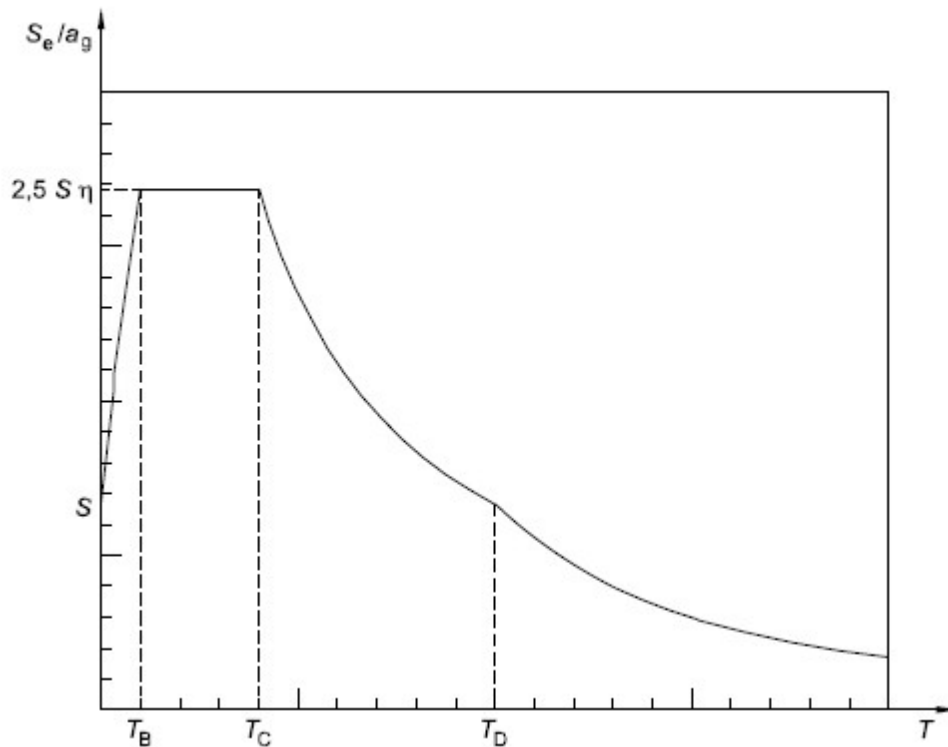
Figure 3.1 - Shape of the elastic response spectrum

2. The values of the periods TB , TC and TD and of the soil factor S describing the shape of the elastic response spectrum depend upon the ground type.

Note 1: The values to be ascribed to TB , TC , TD and S for each ground type and type (shape) of spectrum to be used in a country may be found in its National Annex. If deep geology is not accounted for (see 3.1.2(1)), the recommended choice is the use of two types of spectra: Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined

for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, Ms , not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters S, TB , TC and TD are given in Table 3.2 for the Type 1 Spectrum and in Table 3.3 for the Type 2 Spectrum. Figure 3.2 and Figure 3.3 show
the shapes of the recommended Type 1 and Type 2 spectra, respectively, normalized by a g, for 5% damping. Different spectra may be defined in the National Annex, if deep geology is accounted for.

Ground type	S	$T_B(s)$	$T_C(s)$	$T_D(s)$
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,15	2,0

Table 3.2 - Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	$T_B(s)$	$T_C(s)$	$T_D(s)$
A	1,0	0,05	0,25	1,2
B	1,35	0,05	0,25	1,2
C	1,5	0,10	0,25	1,2
D	1,8	0,10	0,30	1,2
E	1,6	0,05	0,25	1,2

Table 3.3 - Values of the parameters describing the recommended Type 2 elastic response spectra
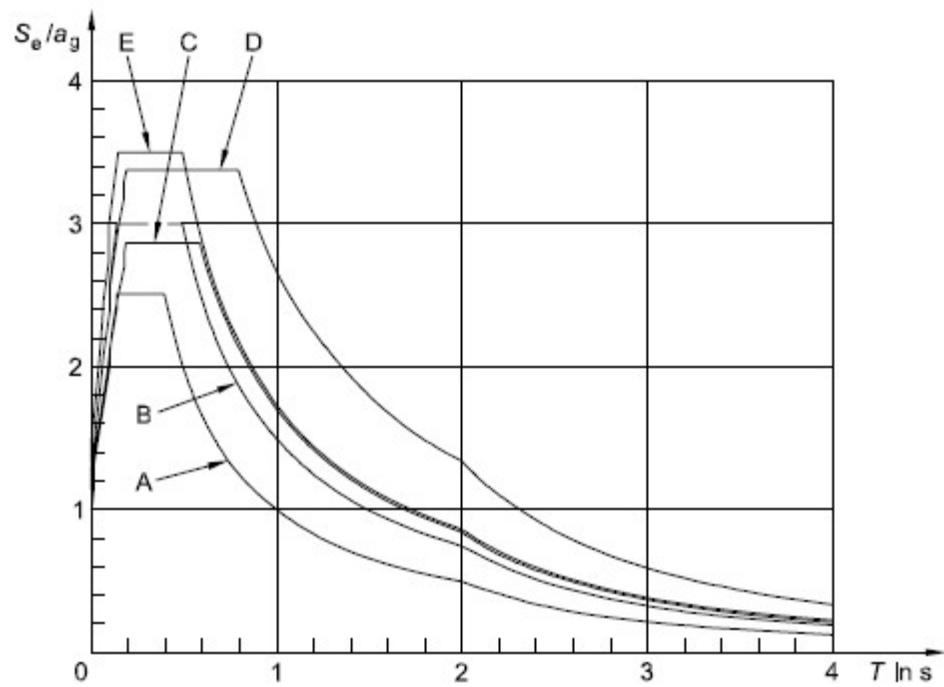
Figure 3.2 - Recommended Type 1 elastic response spectra for ground types A to E (5% damping)
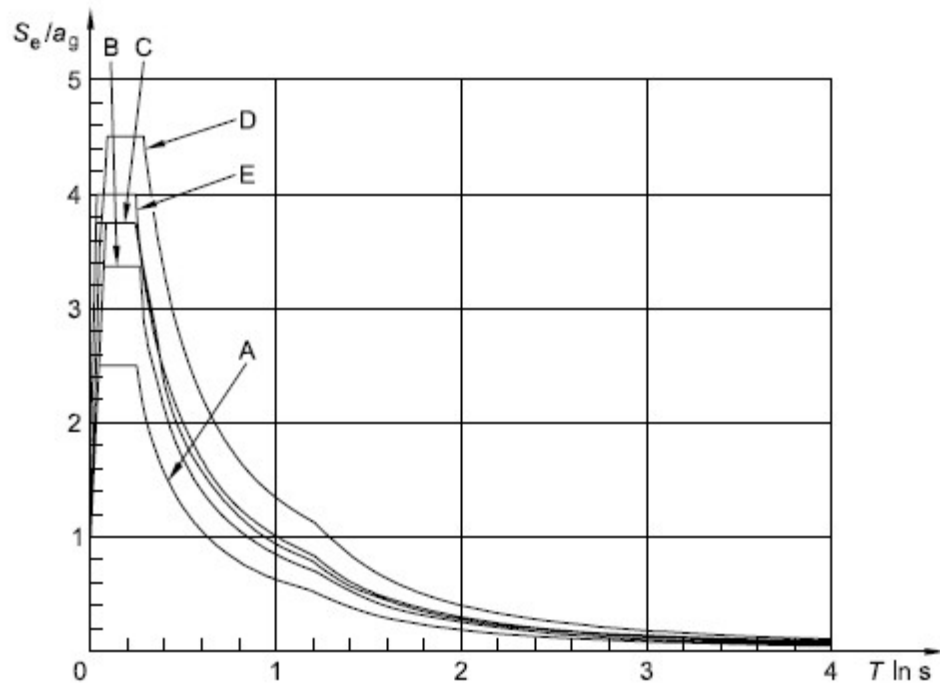
Figure 3.3 - Recommended Type 2 elastic response spectra for ground types A to E (5% damping)

Note 2: For ground types S1 and S2, special studies should provide the corresponding values of S, TB , TC and TD.

3. The value of the damping correction factor   may be determined by the expression:

$$\eta = \sqrt{10/(5+\xi)} \geq 0,55 \qquad (3.6)$$

where:

x is the viscous damping ratio of the structure, expressed as a percentage.

4. If for special cases a viscous damping ratio different from 5% is to be used, this value is given in the relevant Part of EN 1998.

5. The elastic displacement response spectrum, S_{De}(T), shall be obtained by direct transformation of the elastic acceleration response spectrum, S_e(T), using the following expression:

$$S_{De}(T) = S_e(T)\left[\frac{T}{2\pi}\right]^2 \qquad (3.7)$$

6. Expression (3.7) should normally be applied for vibration periods not exceeding 4,0 s. For structures with vibration periods longer than 4,0 s, a more complete definition of the elastic displacement spectrum is possible.

Note: For the Type 1 elastic response spectrum referred to in Note 1 to 3.2.2.2(2)P, such a definition is presented in Informative Annex A in terms of the displacement response spectrum. For periods longer than 4,0 s, the elastic acceleration response spectrum may be derived from the elastic displacement response spectrum by inverting expression (3.7).

3.2.2.3 Vertical elastic response spectrum

1. The vertical component of the seismic action shall be represented by an elastic response spectrum, Sve(T), derived using expressions (3.8)-(3.11).

Note: The values to be ascribed to TB , TC , TD and avg for each type (shape) of vertical spectrum to be used in a country may be found in its National Annex. The recommended choice is the use of two types of vertical spectra: Type 1 and Type 2. As for the spectra defining the horizontal components of the seismic action, if the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, Ms , not greater than 5,5, it is recommended that the Type 2 spectrum is adopted. For the five ground types A, B, C, D and E the recommended values of the parameters describing the vertical spectra are given in Table

*3.4. These recommended values do not apply for special ground types S1
and S2 .*

$$0 \le T \le T_B : S_{ve}(T) = a_{vg} \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot 3,0 - 1) \right] \qquad (3.8)$$

$$T_B \le T \le T_C : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0 \qquad (3.9)$$

$$T_C \le T \le T_D : S_{ve}(T) = a_{vg} \cdot \eta \cdot 3,0 \cdot \left[\frac{T_C}{T} \right] \qquad (3.10)$$

$$T_D \le T \le 4(s): S_{ve}(T) = a_g \cdot \eta \cdot 3,0 \cdot \left[\frac{T_C \cdot T_D}{T^2} \right] \qquad (3.11)$$

Spectrum	a_{vg}/a_g	$T_B(s)$	$T_C(s)$	$T_D(s)$
Type 1	0,90	0,05	0,15	1,0
Type 2	0,45	0,05	0,15	1,0

Table 3.4 - Recommended values of parameters describing the vertical elastic response spectra

2. To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behaviour of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called a ''design spectrum''. This reduction is accomplished by introducing the behaviour factor q.

3. The behaviour factor q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The values of the behaviour factor q, which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes in the various Parts of EN 1998. The value of the behaviour factor q may be different in different horizontal directions of the structure, although the ductility classification shall be the same in all directions.

4. For the horizontal components of the seismic action the design spectrum, Sd(T), shall be defined by the following expressions:

$$0 \le T \le T_B : S_{ve}(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2,5}{q} - \frac{2}{3} \right) \right] \qquad (3.13)$$

$$T_B \le T \le T_C : S_d(T) = a_{vg} \cdot S \cdot \frac{2,5}{q} \qquad (3.14)$$

$$T_C \le T \le T_D : S_d(T) = \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C}{T} \right] \\ \ge \beta \cdot a_g \end{cases} \qquad (3.15)$$

$$T_D \le T : S_d(T) = \begin{cases} = a_g \cdot S \cdot \frac{2,5}{q} \cdot \left[\frac{T_C \cdot T_D}{T^2} \right] \\ \ge \beta \cdot a_g \end{cases} \qquad (3.16)$$

where:
ag, S, TC e TD are as defined in 3.2.2.2;
Sd(T)   is the design spectrum;
q   is the behaviour factor;
          is the lower bound factor for the horizontal design spectrum.

Note: The value to be ascribed to ß for use in a country can be found in its National Annex. The recommended value for   is 0,2.

5. For the vertical component of the seismic action the design spectrum is given by expressions (3.13) to (3.16), with the design ground acceleration in the vertical direction, avg replacing ag , S taken as being equal to 1,0 and the other parameters as defined in 3.2.2.3.
6. For the vertical component of the seismic action a behaviour factor q up to to 1,5 should generally be adopted for all materials and structural systems.
7. The adoption of values for q greater than 1,5 in the vertical direction should be justified through an appropriate analysis.
8. The design spectrum as defined above is not sufficient for the design of structures with base-isolation or energy-dissipation systems.

3.2.3 Alternative representations of the seismic action
3.2.3.1 Time - history representation
3.2.3.1.1 General

1. The seismic motion may also be represented in terms of ground acceleration time-histories and related quantities (velocity and displacement)

2. When a spatial model is required, the seismic motion shall consist of three simultaneously acting accelerograms. The same accelerogram may not be used simultaneously along both horizontal directions. Simplifications are possible in accordance with the relevant Parts of EN 1998.

3. Depending on the nature of the application and on the information actually available, the description of the seismic motion may be made by using artificial accelerograms (see 3.2.3.1.2) and recorded or simulated accelerograms (see 3.2.3.1.3).

3.2.3.1.2 Artificial accelerograms

1. Artificial accelerograms shall be generated so as to match the elastic response spectra given in 3.2.2.2 and 3.2.2.3 for 5% viscous damping (= 5%).

2. The duration of the accelerograms shall be consistent with the magnitude and the other relevant features of the seismic event underlying the establishment of ag .

3. When site-specific data are not available, the minimum duration T s of the stationary part of the accelerograms should be equal to 10 s.

4. The suite of artificial accelerograms should observe the following rules:

   a) a minimum of 3 accelerograms should be used;

   b) the mean of the zero period spectral response acceleration values (calculated from the individual time histories) should not be smaller than the value of agS for the site in question.

   c) in the range of periods between 0,2T1 and 2T1 , where T1 is the fundamental period of the structure in the direction where the accelerogram will be applied; no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping elastic response spectrum.

3.2.3.1.3 Recorded or simulated accelerograms

1. Recorded accelerograms, or accelerograms generated through a physical simulation of source and travel path mechanisms, may be used, provided that the samples used are adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site, and their values are scaled to the value of a agS for the zone under consideration.

2. For soil amplification analyses and for dynamic slope stability verifications see EN 1998-5:2004, 2.2.
3. The suite of recorded or simulated accelerograms to be used should satisfy 3.2.3.1.2(4).

3.2.3.2 Spatial model of the seismic action

1. For structures with special characteristics such that the assumption of the same excitation at all support points cannot reasonably be made, spatial models of the seismic action shall be used (see 3.2.2.1(8)).
2. Such spatial models shall be consistent with the elastic response spectra used for the basic definition of the seismic action in accordance with 3.2.2.2 and 3.2.2.3.

3.2.4 Combinations of the seismic action with other actions

1. The design value Ed of the effects of actions in the seismic design situation shall be determined in accordance with EN 1990:2002, 6.4.3.4.
2. The inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions:

$$\sum G_{k,j} \, ''+'' \sum \Psi_{E,i} \cdot Q_{k,i} \qquad (3.17)$$

where:

$\Psi_{E,i}$ is the combination coefficient for variable action i (see 4.2.4).

3. The combination coefficients $\Psi_{E,i}$ take into account the likelihood of the loads Qk,i not being present over the entire structure during the earthquake. These coefficients may also account for a reduced participation of masses in the motion of the structure due to the non-rigid connection between them.
4. Values of $\Psi_{2,i}$ are given in EN 1990:2002 and values of $\Psi_{E,i}$ other types of structures are given in the relevant parts of EN 1998

4.1.3 Slope stability

4.1.3.3 Methods of analysis (EC 8-part 5)

1. The response of ground slopes to the design earthquake shall be calculated either by means of established methods of dynamic analysis, such as finite elements or rigid block models, or by simplified pseudo-static methods subject to the limitations of (3) and (8) of this subclause.
2. In modelling the mechanical behaviour of the soil media, the softening of the response with increasing strain level, and the possible effects of pore pressure increase under cyclic loading shall be taken into account.

3. The stability verification may be carried out by means of simplified pseudostatic methods where the surface topography and soil stratigraphy do not present very abrupt irregularities.

4. The pseudo-static methods of stability analysis are similar to those indicated in EN 1997-1:2004, 11.5, except for the inclusion of horizontal and vertical inertia forces applied to every portion of the soil mass and to any gravity loads acting on top of the slope.

5. The design seismic inertia forces F_H and FV acting on the ground mass, for the horizontal and vertical directions respectively, in pseudo-static analyses shall be taken as:

$$F_H = 0,5 \; a \; S \; W$$

$$F_V = \pm \, 0,5 \; F_H \text{ if the ratio } a_{vg}/ag \text{ is greater than } 0,6$$

$$F_V = \pm \, 0,33 \; F_H \text{ if the ratio } a_{vg}/a_g \text{ is not greater than } 0,6.$$

Where:

a   is the ratio of the design ground acceleration on type A ground, ag , to the acceleration of gravity g;

a_{vg} is the design ground acceleration in the vertical direction;

a_g is the design ground acceleration for type A ground;

S   is the soil parameter of EN 1998-1:2004, 3.2.2.2;

W   is the weight of the sliding mass.

A topographic amplification factor for a g shall be taken into account according to 4.1.3.2 (2).

6. A limit state condition shall then be checked for the least safe potential slip surface.

7. The serviceability limit state condition may be checked by calculating the permanent displacement of the sliding mass by using a simplified dynamic model consisting of a rigid block sliding against a friction force on the slope. In this model the seismic action should be a time history representation in accordance with 2.2 and based on the design acceleration without reductions.

8. Simplified methods, such as the pseudo-static simplified methods mentioned in (3) to (6) in this subclause, shall not be used for soils capable of developing high pore water pressures or significant degradation of stiffness under cyclic loading.

9. The pore pressure increment should be evaluated using appropriate tests. In the absence of such tests, and for the purpose of preliminary design, it may be estimated through empirical correlations.

2.2 LRFD

2.2.1 LRFD combinations

The LRFD method *(Load Resistance Design Factor)* introduces two types of design coefficients: load factors and resistance factors.

Is is a method that gives importance to the Ultimate Limit State of the structure and does not take into account the concept of "characteristic" resistance.

The method is based on the inequality:

$$Q = \sum \eta_i \gamma_i Q_i < \varphi \cdot R_n$$

where Q is the sum of the nominal loads acting on the structure multiplied by the "load factors" ,ϕ is a "resistance factor" and R_n is the nominal resistance.

Load Combination Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
STRENGTH V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
EXTREME EVENT I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—
EXTREME EVENT II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
SERVICE I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—
SERVICE III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—
FATIGUE— LL, IM & CE ONLY	—	0.75	—	—	—	—	—	—	—	—	—	—	—

Tab. 3.4.1-1 LRFD - Load combinations and load factors

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: **Strength IV only**		**1.50**	**0.90**
DD: Downdrag	**Piles, α Tomlinson Method**	**1.4**	**0.25**
	Piles, λ Method	**1.05**	**0.30**
	Drilled shafts, O'Neill and Reese (*1999*) Method	**1.25**	**0.35**
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Erection Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts		1.95	0.90
• Flexible Metal Box Culverts		1.50	0.90
ES: Earth Surcharge		1.50	0.75

Tab. 3.4.1-2 LRFD - Load factors for permanent loads

WALL-TYPE AND CONDITION		RESISTANCE FACTOR
Nongravity Cantilevered and Anchored Walls		
Axial compressive resistance of vertical elements		Article 10.5 applies
Passive resistance of vertical elements		0.75
Pullout resistance of anchors[1]	• Cohesionless (granular) soils	0.65 [1]
	• Cohesive soils	0.70 [1]
	• Rock	0.50 [1]
Pullout resistance of anchors[2]	• Where proof tests are conducted	1.0 [2]
Tensile resistance of anchor tendon	• Mild steel (e.g., ASTM A 615M bars)	0.90 [3]
	• High strength steel (e.g., ASTM A 722M bars)	0.80 [3]
Flexural capacity of vertical elements		0.90
Mechanically Stabilized Earth Walls		
Bearing resistance		Article 10.5 applies
Sliding		Article 10.5 applies
Tensile resistance of metallic reinforcement and connectors	Strip reinforcements [4]	
	• Static loading	0.75
	• Combined static/earthquake loading	1.00
	Grid reinforcements [4] [5]	
	• Static loading	0.65
	• Combined static/earthquake loading	0.85
Tensile resistance of geosynthetic reinforcement and connectors	• Static loading	0.90
	• Combined static/earthquake loading	1.20
Pullout resistance of tensile reinforcement	• Static loading	0.90
	• Combined static/earthquake loading	1.20
Prefabricated Modular Walls		
Bearing		Article 10.5 applies
Sliding		Article 10.5 applies
Passive resistance		Article 10.5 applies

Tab. 11.5.6-1 LRFD - Resistance factors for retaining works

2.3 Geoapp

Geoapp: the largest web suite for online calculations

The applications present in Geostru Geoapp were created to support the worker for the solution of multiple professional cases.

Geoapp includes over 40 applications for: Engineering, Geology, Geophysics, Hydrology and Hydraulics.

Most of the applications are free, others require a monthly or annual subscription.

Having a subscription means:

- access to the apps from everywhere and every device;
- saving files in cloud and locally;
- reopening files for further elaborations;
- generating prints and graphics;
- notifications about new apps and their inclusion in your subscription;
- access to the newest versions and features;
- support service throught Tickets. Enter topic text here.

2.3.1 Geoapp Section

General and Engineering, Geotechnics and Geology

Among the applications present, a wide range can be used for SPW. For this purpose, the following applications are recommended:

- ➢ Horizontal reaction coefficient of foundation piles
- ➢ Calculation
- ➢ Poles and micropoles
- ➢ Load test
- ➢ Soil classification
- ➢ Newmark

3 Utility

3.1 Conversion Tables

Converting slope inclination into degrees and vice versa

Inclination (%)	Angle (°)	Inclination (%)	Angle (°)
1	0.5729	26	14.5742
2	1.1458	27	15.1096
3	1.7184	28	15.6422
4	2.2906	29	16.1722
5	2.8624	30	16.6992
6	3.4336	31	17.2234
7	4.0042	32	17.7447

8	4.5739	33	18.2629
9	5.1428	34	18.7780
10	5.7106	35	19.2900
11	6.2773	36	19.7989
12	6.8428	37	20.3045
13	7.4069	38	20.8068
14	7.9696	39	21.3058
15	8.5308	40	21.8014
16	9.0903	41	22.2936
17	9.6480	42	22.7824
18	10.2040	43	23.2677
19	10.7580	44	23.7495
20	11.3099	45	24.2277
21	11.8598	46	24.7024
22	12.4074	47	25.1735
23	12.9528	48	25.6410
24	13.4957	49	26.1049
25	14.0362	50	26.5651

Forces conversion

From	To	Operation	Factor
N	kg	Divide by	9.8
kN	kg	Multiply by	102
kN	Tone	Divide by	9.8
kg	N	Multiply by	9.8
kg	kN	Divide by	102
Tone	kN	Multiply by	9.8

1 Newton (N) = 1/9.81 Kg = 0.102 Kg 1 kN = 1000 N

Pressures conversion

From	To	Operation	Factor
$Tons/m^2$	kg/cm^2	Divide by	10
kg/m^2	kg/cm^2	Divide by	10000
Pa	kg/cm^2	Divide by	98000
kPa	kg/cm^2	Divide by	98
Mpa	kg/cm^2	Multiply by	10.2
kPa	kg/m^2	Multiply by	102
Mpa	kg/m^2	Multiply by	102000

1 Pascal (Pa) = 1 Newton/mq 1 kPa = 1000 Pa

3.2   Database of soil physical characteristics

Approximate values of the tangential restitution coefficient (Rt) for the various morphological categories

MORPHOLOGY	R_t
Bedrock	0.87
Outcrops of rock debris	0.85

MORPHOLOGY	R_t
Coarse debris not vegetated	0.85
Average debris not vegetated	0.83
Vegetated debris with shrubs	0.70
Vegetated debris in forest	0.60
Bare soil or lawn	0.55
Paved surfaces	0.90

Approximate values of the normal restitution coefficient (Rn) for the various morphological categories

MORPHOLOGY	R_n
Bedrock	0.40
Outcrops of rock debris	0.38
Coarse debris non-vegetated	0.35
Average debris not vegetated	0.31
Vegetated debris with shrubs	0.30
Vegetated debris in forest	0.28
Bare soil or lawn	0.25
Paved surfaces	0.40

Approximate values of the unit weight in Kg/m^3

Soil	Minimum value	Maximum value
Dry gravel	1800	2000
Wet gravel	1900	2100
Compact dry sand	1700	2000
Compact wet sand	1900	2100
Loose dry sand	1500	1800
Loose wet sand	1600	1900
Sandy clay	1800	2200
Hard clay	2000	2100
Semisolid clay	1900	1950
Soft clay	1800	1850
Peat	1000	1100

Approximate values of the friction angle, in degrees, for soils

Soil	Minimum value	Maximum value
Compact gravel	35	35
Loose gravel	34	35
Compact sand	35	45
Loose sand	25	35
Sandy marl	22	29
Fat marl	16	22
Fat clay	0	30
Sandy clay	16	28

Soil	Minimum value	Maximum value
Silt	20	27

Approximate values of cohesion in Kg/cm²

Soil	Value
Sandy clay	0.20
Soft clay	0.10
Plastic clay	0.25
Semisolid clay	0.50
Solid clay	1
Tenacious clay	2÷10
Compact silt	0.10

Approximate values of the elastic module, in Kg/cm²

Soil	Maximum value of E	Minimum value of E
Very soft clay	153	20.4
Soft clay	255	51
Medium clay	510	153
Hard clay	1020	510
Sandy clay	2550	255
Loess	612	153
Silty sand	204	51
Loose sand	255	102
Compact sand	816	510
Clayey schist	51000	1530
Silt	204	20.4
Loose sand and gravel	1530	510
Compact sand and gravel	2040	1020

Approximate values of the Poisson ratio for soils

Soil	Maximum value of n	Minimum value of n
Saturated clay	0.5	0.4
Not saturated clay	0.3	0.1
Sandy clay	0.3	0.2
Silt	0.35	0.3
Sand	1.0	-0.1
Gravelly sand commonly used	0.4	0.3
Loess	0.3	0.1
Ice	0.36	
Concrete	0.15	

Approximate values of specific weight for some rocks in Kg/m^3

| Rock	Minimum value	Maximum value
Pumice	500	1100
Volcanic tuff	1100	1750
Tufaceous limestone	1120	2000
Coarse sand dry	1400	1500
Fine dry sand	1400	1600
Wet fine sand	1900	2000
Sandstone	1800	2700
Dry clay	2000	2250
Soft limestone	2000	2400
Travertine	2200	2500
Dolomite	2300	2850
Compact limestone	2400	2700
Trachyte	2400	2800
Porphyry	2450	2700
Gneiss	2500	2700
Serpentine	2500	2750
Granite	2550	2900
Marble	2700	2750
Syenite	2700	3000
Diorite	2750	3000
Basalt	2750	3100

4 File conversion from .spw to .spwx

After installing the 2022 version of SPW, in the directory C: \GEOSTRU 2022 is present the folder Convert spw to spwTotxt which contains the executable Spwtotxt.exe that allows you to convert the file with extension .spw in .spwTotxt.

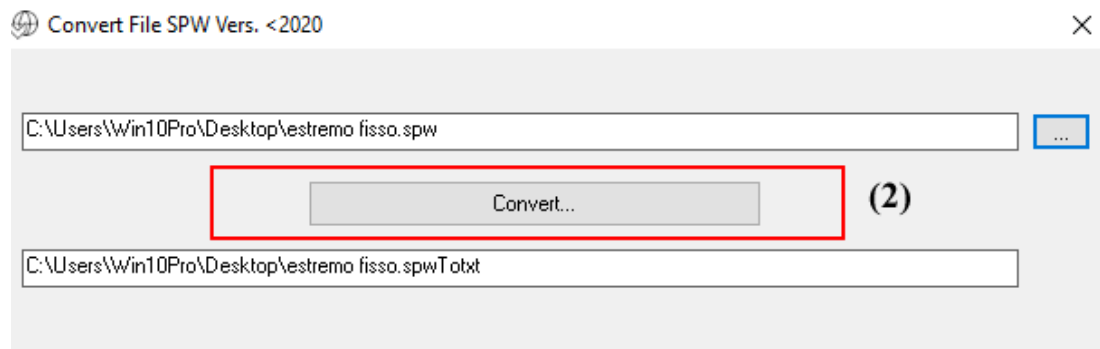Procedure for converting files

Run the executable Spwtotxt.exe, the following image will appear
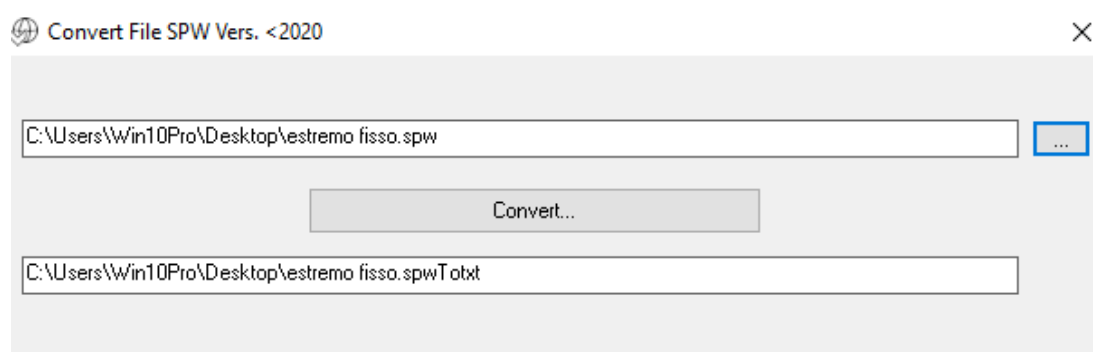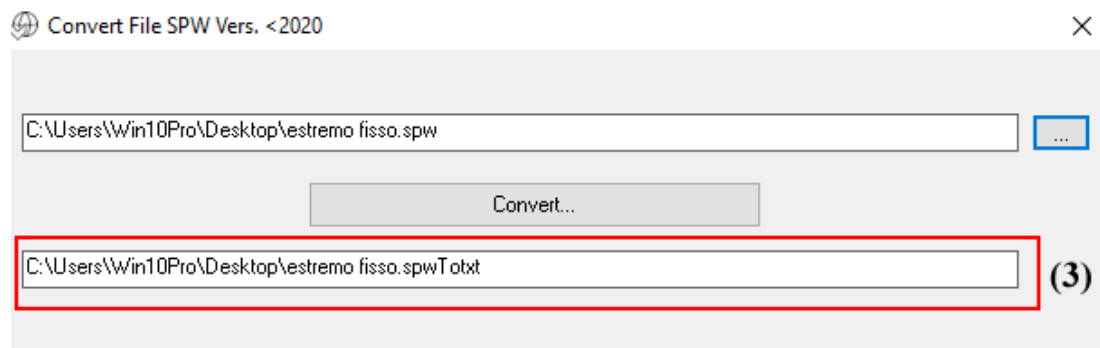

In the field (1) it is necessary to report the path where the file with the .spw extension is located, perform the search by clicking on the […] button. After selecting the file, the Convert command will be activated.
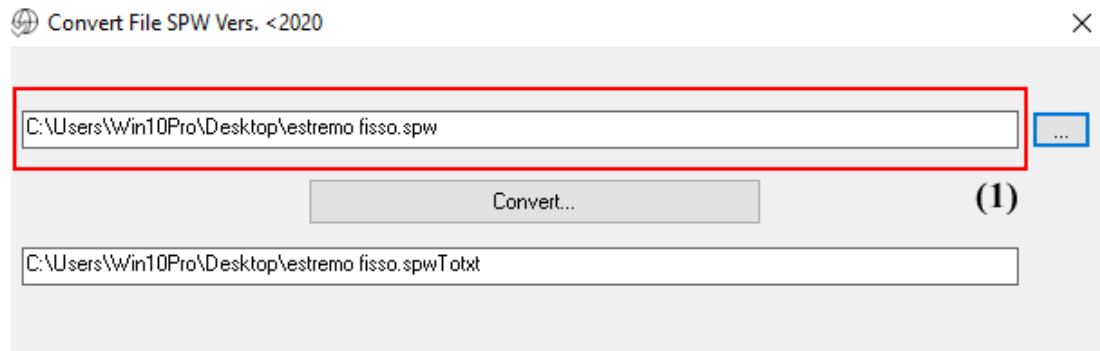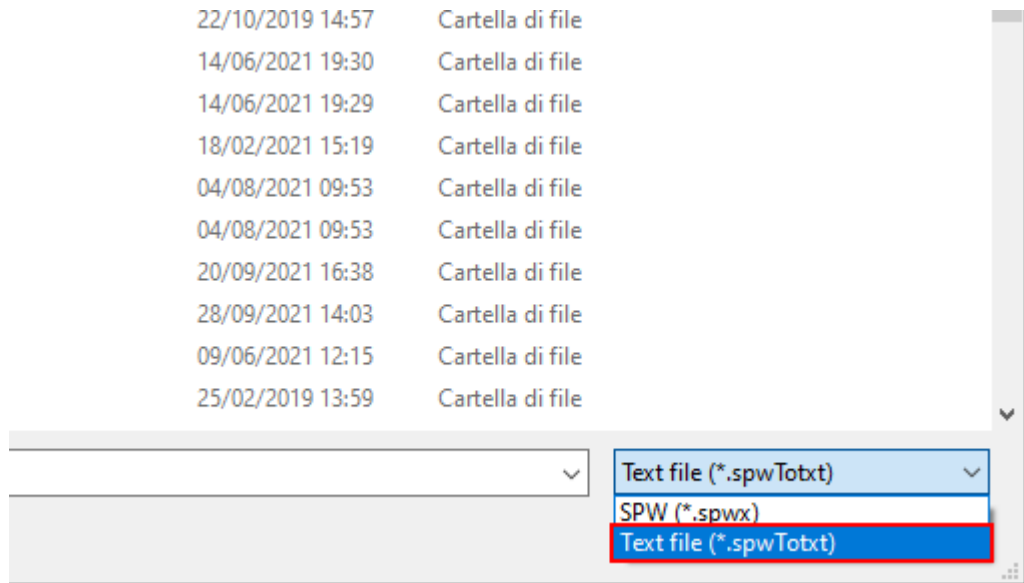
Select the Convert button (2)

The file with extension .spwTotxt will be saved to the directory reported in the field (3).
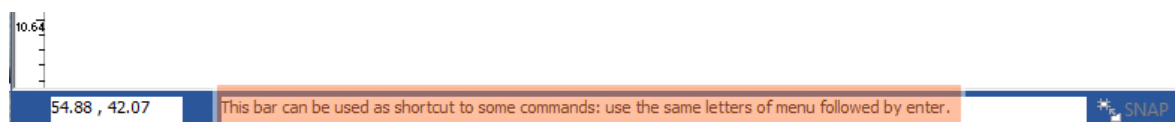

Launch the SPW software, from the File menu select the Open command and choose the extension. spwTotxt select the file generated by the application Spwtotxt.exe. Save the file with extension .spwx.

22/10/2019 14:57	Cartella di file
14/06/2021 19:30	Cartella di file
14/06/2021 19:29	Cartella di file
18/02/2021 15:19	Cartella di file
04/08/2021 09:53	Cartella di file
04/08/2021 09:53	Cartella di file
20/09/2021 16:38	Cartella di file
28/09/2021 14:03	Cartella di file
09/06/2021 12:15	Cartella di file
25/02/2019 13:59	Cartella di file

Text file (*.spwTotxt)

SPW (*.spwx)
Text file (*.spwTotxt)

5 Shortcut commands

The bar shown in figure below can be used for a variety of functionalities:

1) With the shortcut letters of the menu followed by Enter you have quick access to commands.
Ex: N + Enter to create a new file.

2) You can ask a question followed by ? + Enter. In this case an advanced research will be made in the Help manual.
Ex.: Seism+?+Enter for information on seismic analysis.

3) Opening a program in a quick way.
Ex.: Slope+Enter to open GeoStru Slope software.

4) Quick access to GeoStru contacts.
Ex.: Contact+?+Enter to access the contact list.

54.88 , 42.07   This bar can be used as shortcut to some commands: use the same letters of menu followed by enter.   SNAP

Shortcut commands bar

6   Contacts

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